CONSTRUCTIVE TECHNICAL REPORT

On the basic Static and Dynamic calculations of the Object:

Construction of Joint School Gym , "Ramazan Karaj",Nikel

LOCATION : **Nikel, Municipality KRUJE**

CLIENT: **UNDP ALBANIA**

Designer : Edison Drishti

Lic. K. 1566/3

Tirane 2020

CONSTRUCTIVE TECHNICAL REPORT

1. CODES AND REFERENCES

`` Technical Design Condition for Antiseismic Constructions **KTP-N.2-89**``

(ACADEMY OF SCIENCES, Seismological Center)

`` Technical design conditions ``, Volume II, **(KTP-6,7,8,9-1978**)

``**Eurocode 2** : Design of Concrete Structures FINAL DRAFT prEN 1992-1-2``, December 2003)

``**Eurocode 8** : Design of Structures for Earthquake Resistance FINAL DRAFT prEN 1998-1``, December 2003).

2. MATERIALS

The type of concrete provided in the project for the foundations (reinforced concrete connecting beam type) and for all other superstructure elements (columns, reinforced concrete walls, slabs, and beams) is C30 / 37

► The steel used in the facility is imported **S 500** with flow limit σrrj = 500 MPa. This steel type is provided for all types of fittings used in the facility.

► Brick type M-t 150, Mortar type M-ll 50.

► The computational resistances for concrete and steel are obtained by reducing the characteristic resistances according to the class of concrete (or steel) used with the relevant safety factor as follows:

Concrete C30/37 (γ_c=1.5)

Characteristic cubic resistance Rck = 37 N / mm2 Characteristic cylindrical resistance Fck = 30.7N / mm2
Average cylindrical resistance Fcm = Fck + 8 = 38.7 N / mm2
Steel S500 (γ_s=1.15) Computational resistance Fyd = 430 Mpa
Determination of concrete grade is done in accordance with the degree of exposure referred to EN 2061

VLERAT LIMIT TE REKOMANDUARA PER KOMPOZIMIN DHE PERBERJEN E BETONIT

	KLASAT E EKSPOZIMIT																	
	ASNJE RREZIK KORRUDIMI	KORROZIONI I SHKAKTUAR NGA KARBONIZIMI				KORRODIMET NGA KLORURET						NGRIRJA DHE SHKRIRJA				AMBIENTE KIMIKISHT AGRESIVE		
						NGA UJI I DETIT			KLORURE TE TJERA TE NDRYSHME NGA UJI I DETIT									
	X0	XC1	XC2	XC3	XC4	XS1	XS2	XS3	XD1	XD2	XD3	XF1	XF2	XF3	XF4	XA1	XA2	XA3
RAPORTI MAX a/c		0.65	0.60	0.55	0.50	0.50	0.45	0.45	0.55	0.55	0.45	0.55	0.55	0.50	0.45	0.55	0.50	0.45
KLASA MIN E REZISTENCES	C 12/15	C20/25	C 25/30	C30/37	C30/37	C30/37	C35/45	C35/45	C30/37	C30/37	C35/45	C30/37	C25/30	C30/37	C30/37	C30/37	C30/37	C35/45
PERMBAJTJA MIN E CEMENTOS (KG/M3)		260	280	280	300	300	320	340	300	300	320	300	300	320	340	300	320	360
PERMBAJTJA MIN E AJRIT %													4.0[a]	4.0[a]	4.0[a]			
KERKESA TE TJERA												AGREGATE SIPAS EN 12620 ME REZISTENCE TE MJAFTUESHME NE NGRIRJE/SHKRIRJE					CIMENTO REZISTENTE NGA SULFATET	

A) KUR BETONI NUK PERMBAN AJER TE SHTUAR, PERFORMANCA E TIJ DUHET KONFIRMUAR KONFORM SIPAS NJE METODE TE PROVES PERKATESE PER NJE BETON I CILI ESHTE PROVUAR REZISTENCA NE NGRIRJE/SHKRIRJE PER KLASEN RELATIVE TE EKSPOZIMIT

B) NESE PREZENCA E SO24 SJELL KLASEN E EKSPOZIMIT XA2 DHE XA3, ESHTE THELBESORE TE PERDORET NJE CIMENTO REZISTENTE NGA SULFATET. NQS CIMENTOJA ESHTE KLASIFIKUAR E NJE REZISTENCE TE LARTE APO TE MODERUAR NGA SULFATET, CIMENTOJA DUHET TE PERDORET ME KLASE EKSPOZIMI XA2 (DHE TE NJE KLASE TE EKSPOZIMI XA1 NESE ESHTE E APLIKUESHME) DHE CIMENTOJA ME REZISTENCE TE LARTE NGA SULFATET DUHET TE PERDORET ME KLASE EKSPOZIMI XA3.

KLASA	ULJA NGA KONI
S1	NGA 10 DER NE 20
S2	NGA 50 DERI NE 90
S3	NGA 100 DERI NE 150
S4	NGA 160 DERI NE 210
S5	>220

KLASA E EKSPOZIMIT NE AMBIENT	SPESORI MINIMAL I SHTRESES MBROJTESE			
	KOHA E NEVOJSHME 50 VJET		KOHA E NEVOJSHME 100 VITE	
	C.A	C.A.P	C.A	C.A.P
X0	10	10	20	20
XC1	15	25	25	35
XC2.XC3	25	35	35	45
XC4	30	40	40	50
XS1,XD1	35	45	45	55
XS2,XD2	40	50	50	60
XS3,XD3	45	55	55	65

3. COMPUTER ANALYSIS AND CALCULATION

Static and dynamic analysis to determine the response of the structure to different types of loading of the structure was performed with the program HOLO bim V 9.32. The modeling of the structure as a whole and of each element is done on the basis of the finite element methodology (Finite Element Method - FEM) which is a rough and practical method finding wide use today in terms of the superiority created by the use of computer programs. .

The structure analysis is performed using the method of Limit state in full compliance with Eurocodes.

There are two different situation of limit state, Ultimate limit state (SLU) and Serviciability limit state (SLS)

Dynamic analysis is based on modal analysis with the reaction spectrum method. The calculated dynamic (seismic) loads are accepted as equivalent static loads and are applied in place of concentrated masses. The basis for the method of dynamic calculations with the method of reaction spectrum is the analysis of its own values and its own vectors. By this method the forms of self-oscillations and the frequencies of free oscillations are determined. The values and vectors themselves undoubtedly provide a clear and complete picture of the behavior of the structure under the action of dynamic loads. The program HOLO bim automatically requires modes with lower circular frequencies (higher periods). The maximum number of modes required by the program is conditioned by the constructor himself (in the case of the object in question n = 9 modes), while the floor measures of this object are considered with three degrees of freedom, of which 2 rotating and one translational according to the plan of the soleta itself. The cyclic frequency f (cycles / sec), the circular frequency ω (rad / sec) and the period T (sec) are related to each other through the relations: $T = 1 / f$ and $f = \omega / 2\pi$. The result of the analysis is the displacements, internal forces (M, Q, N,) and the stresses σ in each element of the structure.

4 CALCULATING LOADS ON THE PROJECT

4.1 Loads acting on structures

For the building under study the actions of the following factors have been taken into account:

- Dead Loads-DL and Live Loads-LL

-Wind

-Siesmicity

1- Faktoret Veprues mbi Strukture (Aksionet)

Characteristic operators (loads, thermal variations, distortions, torsions, etc.) are determined in accordance with EC1. based on the following classifications:

1. Classification of actions based on the manner of their execute

a) **Direct** : which can be concentrated forces or distributed loads, can be fixed or movable.
b) **Indirect** : which can be displacements, distortions, change of temperature, humidity, internal pressure, yielding of supports, etc.
c) **Degradation**: which can be:
endogenous when we have a natural change of the material of which the structure is composed or exogenous when the material loses its characteristic properties under the influence of external agents.

2. Classification of actions based on structural response

a) **Static** : Actions that when applied to the structure do not provoke significant acceleration in the whole structure or in specific parts of it.
b) **Pseudo Static** : dynamic actions that are presented as an equivalent static action
c) **Dynamic**: actions that cause significant accelerations in the structure itself or in particular parts of it.

3. Classification of actions based on their variation in time

a) **Permanent (G)** : Actions that are exercised throughout the nominal life of the structure and that the variation of their intensity over time is so easy that the actions can be considered constant over time, here are:
Structural permanent (G1)
the own weight of all the elements of the structure
the specific gravity of the terrain when it affects the structure
forces exerted by the terrain (excluding variable loads exerted on the ground)
forces exerted by water pressure (when they are configured as constant in time)
Permanent non-structural (G2)
the own weight of all non-structural elements
actions from displacements and deformations foreseen in the project
Pretensioning, compression (P)
Attraction – viscosity
Differential displacements

b) **Variable (Q)** : actions that are exerted on a structure or on a particular element of it immediately and that result in significantly different values at times which can be:
long-term: actions that exert a considerable intensity although not permanently but that have a not insignificant duration compared to the nominal life of the structure.
short-term: actions that exert a considerable intensity but have a short duration compared to the nominal life of the structure
c) Accidental (A): actions that are verified in exceptional cases during the nominal life of the structure:
in case of fire
in case of explosion
in case of shock or collision

d) **Siecmic (E)**: actions that derive during earthquakes

4.2 Permanent Loads (Dead Loads-DL)

Permanent loads mean: Own weight of all structural elements (foundations, beams, columns, slabs,staircases etc.) which are automatically calculated by the program, as well as own weight of non-structural elements (floor layers, brick partition walls with bars, balcony railings, stair treads, etc.). The normalized weights of the materials taken into account in the load calculation are taken as follows:

Specific weight of concrete:	25.00 kN/m3
Own weight of slab:	1.50 kN/m²
Specific weight of steel:	78.00 kN/m3
Load of tile layers:	1.50 kN/m²
Load of perimeter walls:	3.60 kN/m²
Stairs cladding layers:	1.30 kN/m²
Partition wall load:	2.10 kN/m²
Soil specific gravity:	18.00 kN/m3

4.3 Temporary Loads (Live Loads-LL)

As temporary loads in the structure are calculated the loads of use of floors, stairs, balconies, terraces, etc., as defined in EN 1991-1-1: 2001 (Page 21-22) Table 6.2 (Table 6.2 - Imposed loads on floors , balconies and stairs in buildings) for categories C1, C3.

For all school spaces the temporary load is taken into account with a maximum value of 5 kN / m (in favor of safety).

4.4 Combination of actions (Loads)

Starting from a single load action the program generates a series of load schemes called Basic Load Conditions, which are then combined with each other for different load operations in such a way that: they result as the most unfavorable, based on the duration, frequency and low probability of simultaneous action of all loads with the most unfavorable value.

In accordance with the Eurocodes, the following load combinations are generally considered depending on the limit state ULS and ELS:

Static Combination : ULS (Basic or Fundamental Combination)

ELS (Characteristic (rare), Frequent, Almost Permanent)

Siesmic Combination : ULS

ELS

Defined as the characteristic value Qk of a variable action, the value corresponding to the maximum value of a fragment that includes 95% of the possible cases in relation to a reference time period of this action.In determining the combination of actions that can act simultaneously in the structure, the term Qkj - indicates the variable actions in combination, while Qk1 - indicates the dominant variable actionQk2, Qk3 .... are given actions that can act simultaneously with what is dominant

The variable actions Qkj are combined with the coefficients ψ0j, ψ1j and ψ2j which refer to the duration in percentage in relation to the intensity of the variable action. These are defined as follows:

ψ2j x Qkj -almost permanent value: determines the average value of the time distribution of the intensity of the variable action

ψ1j x Qkj - frequent value: determines the corresponding value of a fragment of 95% of the time distribution of the intensity of the variable action that can be exceeded only for a fraction of 5% of the reference period.

ψ0j x Qkj - rare value: determines the low value of the temporal distribution of the intensity of the variable action but considerable in the possibility of interaction with other variable actions

The values of the combination coefficients for residential buildings and social and industrial buildings are given in the table

Kategoria		ψ0j	ψ1j	ψ2j
A	Ambiente Banimi	0.7	0.5	0.3
B	Zyra	0.7	0.5	0.3
C	Godina qe kane popullim	0.7	0.7	0.6
D	Godina komerciale	0.7	0.7	0.6
E	Bibloteka,arshiva,magazina,industriale	1.0	0.9	0.8
F	Parkime (auto me peshe deri 30kN)	0.7	0.7	0.6
G	Parkime (auto me peshe mbi 30kN)	0.7	0.5	0.3
H	Mbulesa	0.0	0.0	0.0
Era		0.6	0.2	0.0
Debora	kuota deri 1000m nga niveli detit	0.5	0.2	0.0
Debora	kuota mbi 1000m nga niveli detit	0.7	0.5	0.2
Temperatura		0.6	0.5	0.0

In order to perform the controls in the limit states these combinations of actions are defined

STATIC Combination

1. Basic Combination, generally used for the ultimate limit statee (ULS):

$$F_d = \gamma_g \cdot G_k + \gamma_{Q1} \cdot Q_{k1} + \sum_{i=2}^{n} \gamma_{Qi} \cdot (\Psi_{0i} \cdot Q_{ki})$$

γ_g - amplification coefficient for the action of permanent loads

γ_{Qi}- amplification coefficient for the action of variable loads

2. Characteristic (rare) combination, generally used for the first limit state or as it is otherwise called the serviceability limit state (ELS) - non-reversible used in controls which are performed with Allowed tension (TA):

$$F_d = G_k + Q_{k1} + \sum_{i=2}^{n} (\Psi_{0i} \cdot Q_{ki})$$

3. Frequent Combination, generally used for reversible serviceability limit state (ELS):

$$\mathbf{F_d = G_k + \Psi_{11} \cdot Q_{k1} + \sum_{i=2}^{n} \cdot (\Psi_{2i} \cdot Q_{ki})}$$

4. Almost Permanent combination, used for serviceability limit state (ELS) - in case of prolonged effect operations:

$$\mathbf{F_d = G_k + \sum_{i=1}^{n} \cdot (\Psi_{2i} \cdot Q_{ki})}$$

Below we give the matrix of combination coefficients in the most general case when the structure is under the action of loads of slabs, snow, and temperature:

ULS Kombinimi Baze							
Vlerat e projektit per ngarkesat				Vlerat e kombinimit			
perm.	soleta	bora	temp	perm.	soleta	bora	temp
G_k	Q_{k1}	Q_{k2}	Q_{k3}	Υ_G*G_k	Υ_Q*Q_{k1}	Υ_Q*Q_{k2}	Υ_Q*Q_{k3}
				Υ_G*G_k	$\Upsilon_Q*\Psi_{01}*Q_{k1}$	$\Upsilon_Q*\Psi_{01}*Q_{k2}$	$\Upsilon_Q*\Psi_{01}*Q_{k3}$
				Υ_G*G_k	$\Upsilon_Q*\Psi_{01}*Q_{k1}$	$\Upsilon_Q*\Psi_{01}*Q_{k2}$	$\Upsilon_Q*\Psi_{01}*Q_{k3}$
				Υ_G*G_k	$\Upsilon_Q*\Psi_{01}*Q_{k1}$	$\Upsilon_Q*\Psi_{01}*Q_{k2}$	$\Upsilon_Q*\Psi_{01}*Q_{k3}$

ELS Kombinimi Rralle							
Vlerat e projektit per ngarkesat				Vlerat e kombinimit			
perm.	soleta	bora	temp	perm.	soleta	bora	temp
G_k	Q_{k1}	Q_{k2}	Q_{k3}	G_k	Q_{k1}	$\Psi_{02}*Q_{k2}$	$\Psi_{03}*Q_{k3}$
				G_k	$\Upsilon_Q*\Psi_{01}*Q_{k1}$	Q_{k2}	$\Psi_{03}*Q_{k3}$
				G_k	$\Upsilon_Q*\Psi_{01}*Q_{k1}$	$\Psi_{02}*Q_{k2}$	Q_{k3}
				G_k	$\Upsilon_Q*\Psi_{01}*Q_{k1}$	$\Psi_{02}*Q_{k2}$	$\Psi_{03}*Q_{k3}$

ELS Kombinimi Shpeshte							
Vlerat e projektit per ngarkesat				Vlerat e kombinimit			
perm.	soleta	bora	temp	perm.	Sherbimi	bora	temp
G_k	Q_{k1}	Q_{k2}	Q_{k3}	G_k	$\Psi_{11}*Q_{k1}$	$\Psi_{22}*Q_{k2}$	0
				G_k	$\Psi_{21}*Q_{k1}$	$\Psi_{12}*Q_{k2}$	0
				G_k	$\Psi_{21}*Q_{k1}$	$\Psi_{22}*Q_{k2}$	$\Psi_{12}*Q_{k3}$
				G_k	$\Psi_{21}*Q_{k1}$	$\Psi_{22}*Q_{k2}$	0

ULS Kombinimi Gati Permanent							
Vlerat e projektit per ngarkesat				Vlerat e kombinimit			
perm.	soleta	bora	temp	perm.	Sherbimi	bora	temp
G_k	Q_{k1}	Q_{k2}	Q_{k3}	G_k	$\Psi_{21}*Q_{k1}$	$\Psi_{22}*Q_{k2}$	Υ_Q*Q_{k3}

SIESMIC Combination

In the case of seismic combinations compared to static ones, variable actions are considered in both limit states through their almost permanent values and neither of them is considered dominant. It is only the shape of the seismic spectrum that makes the difference, so what differs from ULS to ELS is the value of the seismic action.

The general form of seismic combination required to evaluate the effect of seismic action and all other accompanying actions is

$$F_d = G_k + \sum_{i=1}^{n} (\Psi_{2i} \cdot Q_{ki}) \pm E_k$$

Below we are giving the combination matrix belonging to ULS and ELS in the presence of seismicity. For seismic ultimate limit state (usually called ULS-Seismic) and for any direction of seismic action (Ex, Ey) given the double (+, -) eccentricity as well as the two possible directions of displacement of the center of mass (ex , ey) four combinations will be obtained.

For each of these four combinations should be considered four possible combinations due to the simultaneous presence of seismic action in both directions.

In this way 16 elementary combinations are obtained for each direction of seismic action and in total we have 32 combinations for the seismic boundary condition.

Kombinimi I Ngarkesave						Ngarkesat Elementare					
Perm	Variabel	Veprimi Sizmik				G	Σ Ψ_{2j}*Q_{kj}	Ex	ey	Ey	ex
G_k	Σ Ψ_{2j}*Q_{kj}	Ex	ey	0.3*E_y	+ e_x	1.0	1.0	1.0	1.0	0.3	0.3
					- e_x	1.0	1.0	1.0	1.0	0.3	-0.3
				-0.3*E_y	+ e_x	1.0	1.0	1.0	1.0	-0.3	0.3
					- e_x	1.0	1.0	1.0	1.0	-0.3	-0.3
			-ey	0.3*E_y	+ e_x	1.0	1.0	1.0	- 1.0	0.3	0.3
					- e_x	1.0	1.0	1.0	- 1.0	0.3	-0.3
				-0.3*E_y	+ e_x	1.0	1.0	1.0	- 1.0	-0.3	0.3
					- e_x	1.0	1.0	1.0	- 1.0	-0.3	-0.3
		- Ex	ey	0.3*E_y	+ e_x	1.0	1.0	- 1.0	1.0	0.3	0.3
					- e_x	1.0	1.0	- 1.0	1.0	0.3	-0.3
				-0.3*E_y	+ e_x	1.0	1.0	- 1.0	1.0	-0.3	0.3
					- e_x	1.0	1.0	- 1.0	1.0	-0.3	-0.3
			-ey	0.3*E_y	+ e_x	1.0	1.0	- 1.0	- 1.0	0.3	0.3
					- e_x	1.0	1.0	- 1.0	- 1.0	0.3	-0.3
				-0.3*E_y	+ e_x	1.0	1.0	- 1.0	- 1.0	-0.3	0.3
					- e_x	1.0	1.0	- 1.0	- 1.0	-0.3	-0.3

Kombinimi I Ngarkesave						Ngarkesat Elementare					
Perm	Variabel	Veprimi Sizmik				G	Σ Ψ_{2j}*Q_{kj}	Ex	ey	Ey	ex
G_k	Σ Ψ_{2j}*Q_{kj}	Ey	ex	0.3*E_x	+ e_y	1.0	1.0	1.0	1.0	0.3	0.3
					- e_y	1.0	1.0	1.0	1.0	0.3	-0.3
				-0.3*E_x	+ e_y	1.0	1.0	1.0	1.0	-0.3	0.3
					- e_y	1.0	1.0	1.0	1.0	-0.3	-0.3
			-ex	0.3*E_x	+ e_y	1.0	1.0	1.0	- 1.0	0.3	0.3
					- e_y	1.0	1.0	1.0	- 1.0	0.3	-0.3
				-0.3*E_x	+ e_y	1.0	1.0	1.0	- 1.0	-0.3	0.3
					- e_y	1.0	1.0	1.0	- 1.0	-0.3	-0.3
		- Ey	ex	0.3*E_x	+ e_y	1.0	1.0	- 1.0	1.0	0.3	0.3
					- e_y	1.0	1.0	- 1.0	1.0	0.3	-0.3
				-0.3*E_x	+ e_y	1.0	1.0	- 1.0	1.0	-0.3	0.3
					- e_y	1.0	1.0	- 1.0	1.0	-0.3	-0.3
			-ex	0.3*E_x	+ e_y	1.0	1.0	- 1.0	- 1.0	0.3	0.3
					- e_y	1.0	1.0	- 1.0	- 1.0	0.3	-0.3
				-0.3*E_x	+ e_y	1.0	1.0	- 1.0	- 1.0	-0.3	0.3
					- e_y	1.0	1.0	- 1.0	- 1.0	-0.3	-0.3

5. SEISMIC CONSIDERATIONS

5.1 Siesmic parameters

Siesmic Zone: **III (α_{gR} =0.27)**
Ground Type: **D**
Building importance Class: **III**
Seismic action directions: **X, Y**
Elastic response spectrum: **Type 1**
Ductility class: **(DCM)**

5.2 Classification according to the type of structural system *[EC8 §5.2.2.1]*

From the calculations made the building is classified as Frame system in both directions
In x direction: Frame system
In the y direction: Frame system

$V_{x/y}$ Shear strength by direction X-X/Y-Y

Data:

	V_x	V_y
12	14.22	6.54
4	11.73	14.23
7	18.98	5.11
6	18.73	5.13
15	4.47	9.70
11	13.58	2.09
3	11.52	24.20
17	1.45	3.52
16	4.31	9.36
10	14.20	6.48
1	11.73	14.24
9	1.21	10.39
5	18.97	5.10
14	4.31	9.33
8	1.21	10.41
2	11.52	24.25
13	1.45	3.52

6. REGULATION CRITERIA

6.1 Regularity in the plan *[EC8 §4.2.3.2]*

The building is not considered regular in the plan

6.2 Regularity in the height *[EC8 §4.2.3.3]*

The building is not considered regular in height

7. BEHAVIOUR FACTOR CALCULATION *[EC8 §5.2.2.2]*

Symbols:

q behaviour factor
q_o the basic value of the behaviour factor
k_w factor reflecting the prevailing failure mode in structural systems with walls
α_1 multiplier of horizontal seismic design action at formation of first plastic hinge in the system
α_u multiplier of horizontal seismic design action at formation of global plastic mechanism

Data :

Structural system by direction X-X	Frame System
Structural system by direction Y-Y	Frame System
Ductility Class	DCM
Regularity in the plan	NO
Regularity in the height	NO

	α_u/α_1	α_{qo}	q_o	k_w	q
Direction X-X	1.30	3.00	2.76	1.00	2.76
Direction Y-Y	1.30	3.00	2.76	1.00	2.76

Result: Behaviour Factor *q*: **2.76**

8. DETERMINATION OF PROJECT SPECTRUM *[EC8 §3.2.2]*

Symbols:

α_{gR} reference peak ground acceleration on type A ground
γ_i importance factor of the building
q behavior factor
S soil factor obtained from geotechnical data (*table 3.2 dhe 3.3 EN 1988-1*)
T vibration period of a linear single degree of freedom system
ξ viscous damping ratio on %
β lower bound factor for the horizontal design spectrum
$S_d(T)$ design spectrum
g gravity accleration

Data :

γ_i 1.20 (III)
ξ 5 %
β 0.20

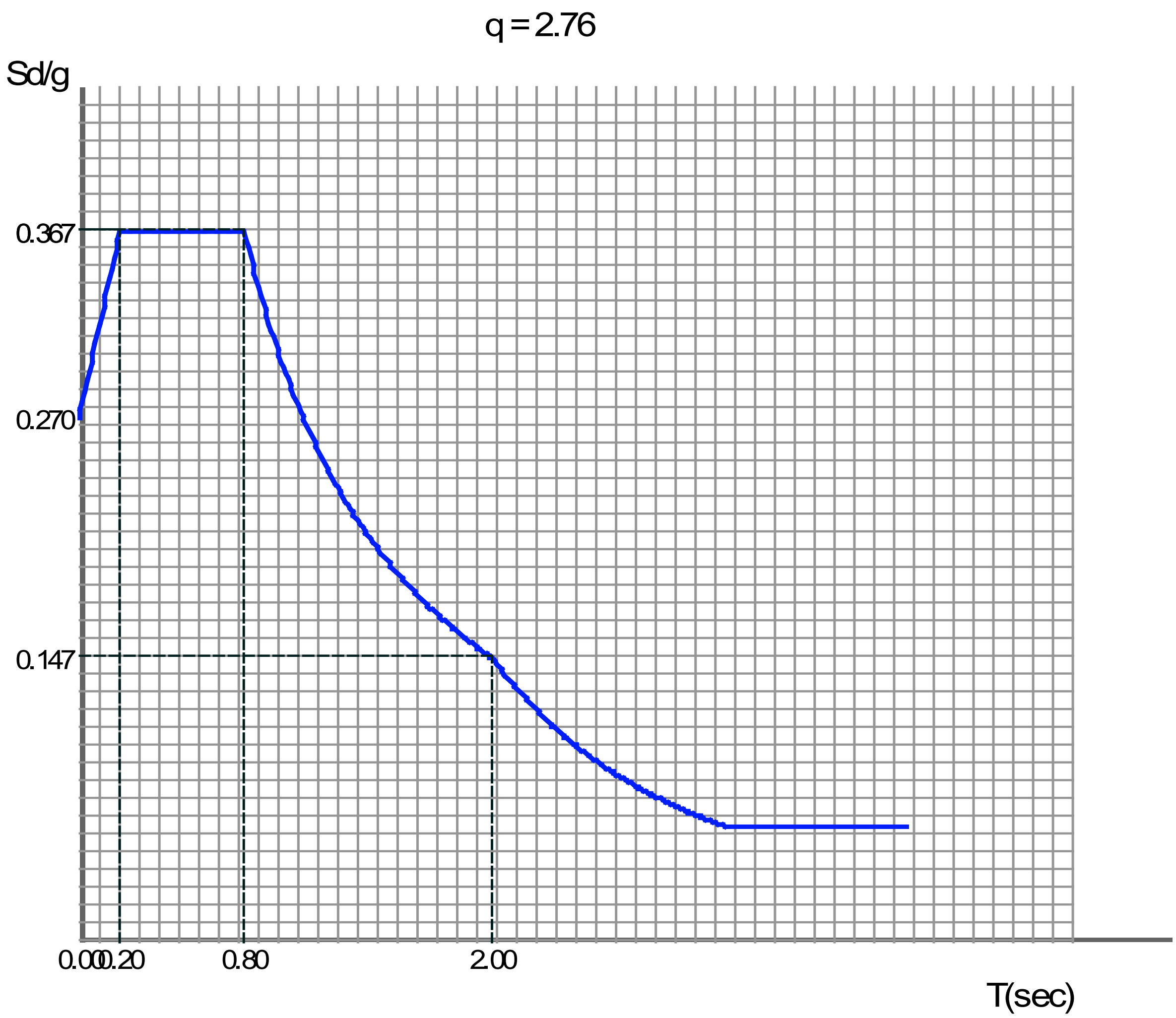
Spectrum : Type 1 (Ms >5.5)
Ground Type: D

	α_{gR} (III)	q	S	T_B (s)	T_C (s)	T_D (s)
horizontal	0.25	2.76	1.35	0.20	0.80	2.00
vertical	0.23	1.50	1.00	0.05	0.15	1.00

Results:

S_D/g

	0	T_B	T_C	T_D	4
horizontal	0.27	0.37	0.37	0.15	0.06
vertical	0.15	0.18	0.05	0.05	0.05


q=2.76
Sd/g
0.367
0.270
0.147
0.00
0.20
0.80
2.00
T(sec)

9. MODAL RESPONSE SPECTRUM ANALYSIS *[EC8 §4.3.3.3]*

9.1 Eigenvalue analysis

Modal Shape Table:

Shape	Ω (rad/sec)	T (sec)	S_d	Ψ_x	C_x (%)	Ψ_y	C_y (%)	Ψ_z	C_z (%)
1	13.01	0.482974	3.67	-0.15	0.01	11.92	35.73	-0.03	0.00
2	13.73	0.457536	3.67	10.27	26.54	0.22	0.01	0.00	0.00
3	15.34	0.409534	3.67	0.08	0.00	-6.89	11.95	-0.02	0.00
4	18.93	0.331863	3.67	-2.84	2.03	-0.01	0.00	0.00	0.00
5	22.99	0.273242	3.67	6.41	10.35	0.00	0.00	0.00	0.00
6	24.68	0.254574	3.67	0.00	0.00	-11.62	33.99	-0.42	0.03
7	28.15	0.223202	3.67	8.00	16.10	0.00	0.00	0.00	0.00
8	29.99	0.209504	3.67	-0.26	0.02	0.02	0.00	-0.03	0.00
9	30.14	0.208487	3.67	1.86	0.87	0.02	0.00	0.00	0.00
10	31.39	0.200151	3.67	10.90	29.89	-0.05	0.00	-0.01	0.00
11	31.57	0.199033	3.66	-0.27	0.02	-2.12	1.13	-0.24	0.01
12	35.90	0.175042	3.55	0.63	0.10	0.00	0.00	0.00	0.00
13	39.23	0.160169	3.48	-0.01	0.00	-0.55	0.08	0.02	0.00
14	43.46	0.144579	3.40	-0.02	0.00	-0.27	0.02	-0.07	0.00
SUM					**85.93**		**82.91**		

of 90% sum of the effective modal shapes

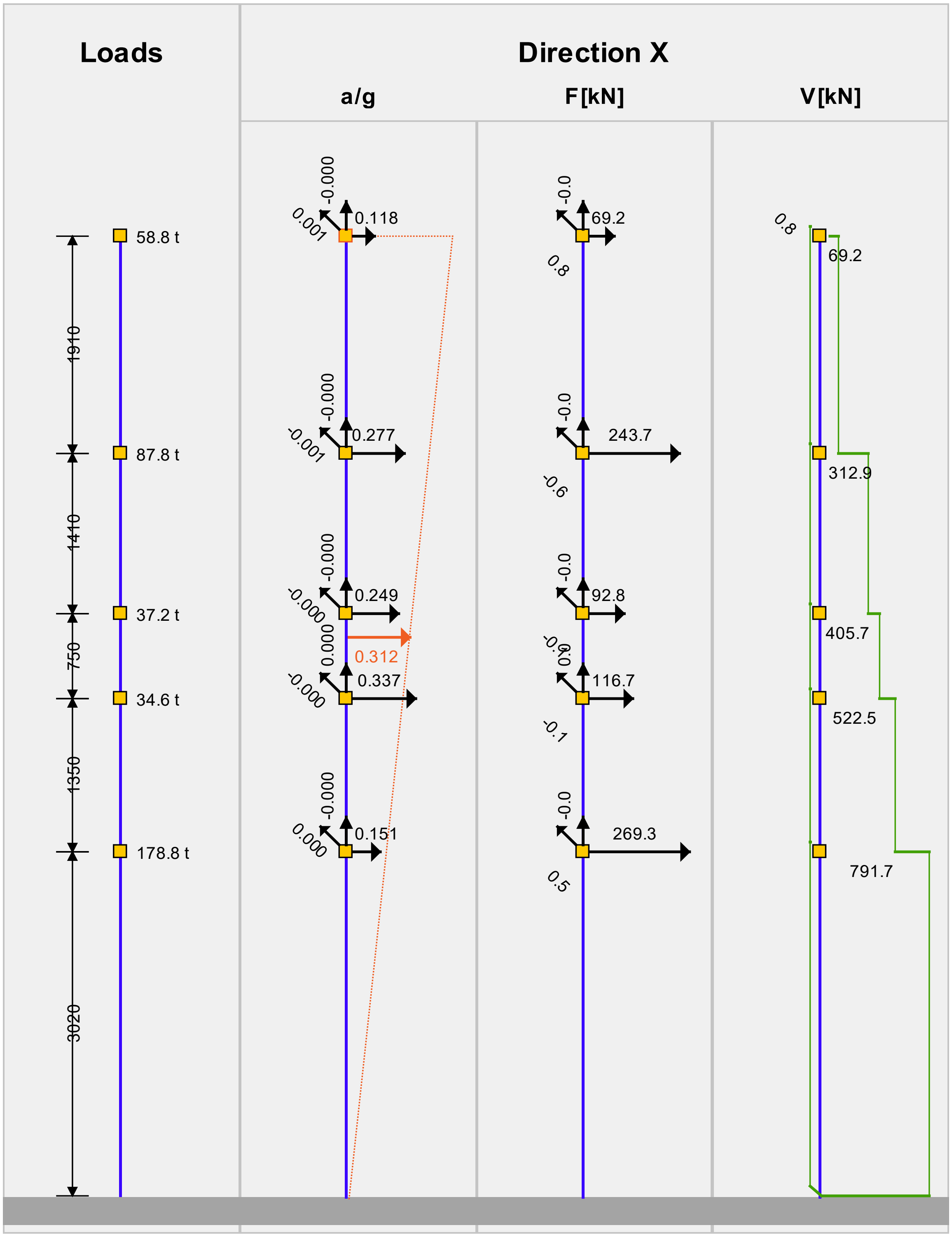
Dir.	k		$3n^{0.5}$	$T_k \leq 0.20s$
x	14	≥	6.71	0.145 ≤ 0.20
y	14	≥	6.71	0.145 ≤ 0.20

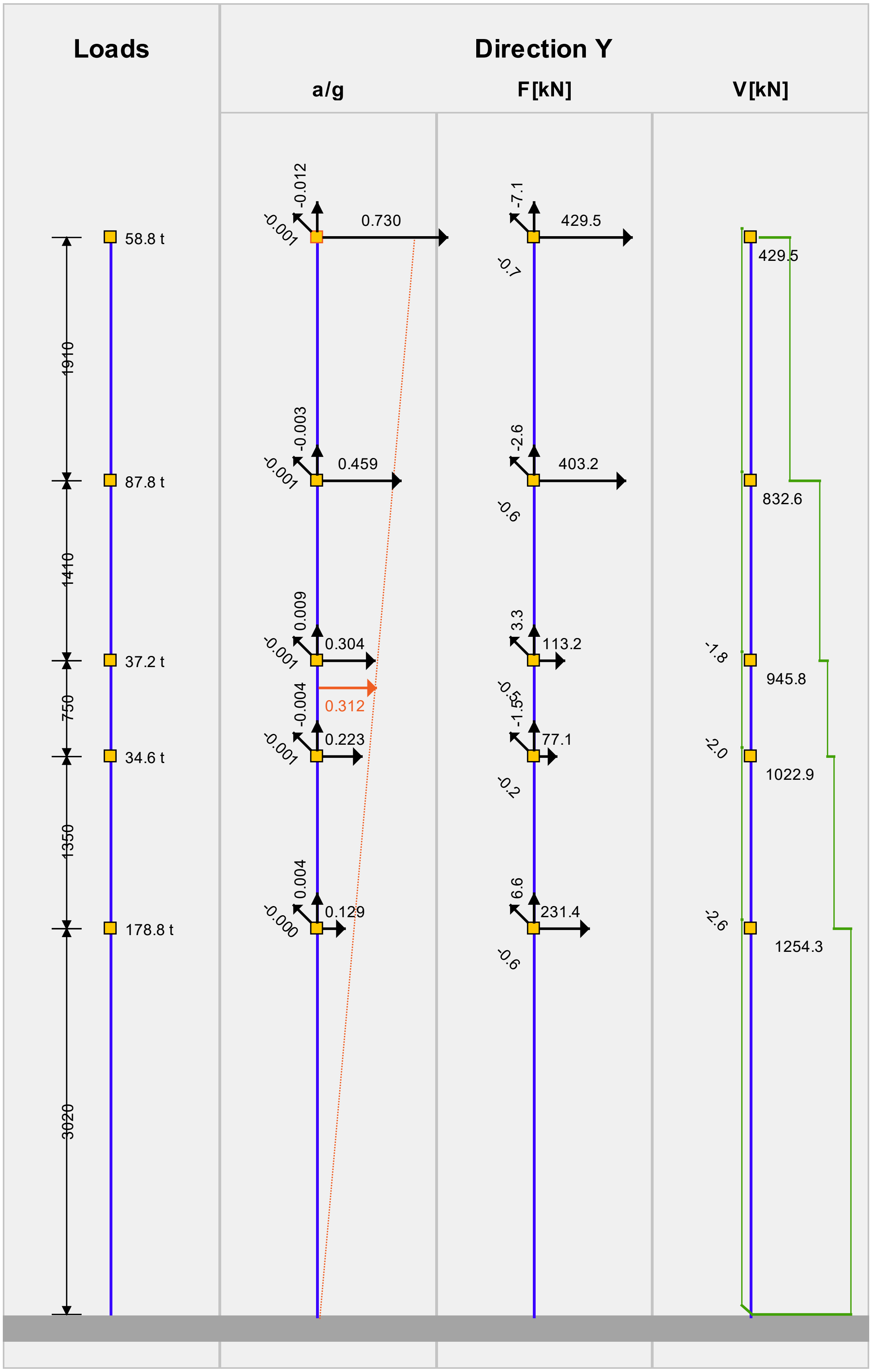
k, is the number of modes taken into account
n, is the number of storeys above the foundation or the top of a rigid basement
Tk, is the period of vibration of mode k

a_{CM} = 2.9218

Verification : Σ(M) = 397.4032, Σ(V) = 792.2008, a = 1.9934

9.2 **Calculation of seismic accelerations and forces**




Loads
Direction Y
a/g
F[kN]
V[kN]
58.8 t
87.8 t
37.2 t
34.6 t
178.8 t
1910
1410
750
1350
3020
-0.012
-0.001
0.730
-0.003
0.459
0.009
0.304
0.312
-0.004
0.223
0.004
-0.000
0.129
-7.1
429.5
-0.7
-2.6
403.2
-0.6
3.3
113.2
-0.5
-1.5
77.1
-0.2
6.6
231.4
832.6
-1.8
945.8
-2.0
1022.9
-2.6
1254.3

The elements of the structure are also checked in accordance with the allowable deformations caused in them by the action of normative loads. In these combinations the load combination coefficients are accepted as units.

The effect of accidental torsion is included in the calculation of the building being automatically incorporated into the level of seismic forces. The eccentricity of the action of seismic forces for each floor is accepted 5% of the dimension of the building perpendicular to the seismic direction in the study.

10. FOUNDATIONS

The design of the foundations is done in harmony with the structure that will support it from above, in accordance with the geotechnical conditions of the terrain as well as the general requirements of the building.

The basic control criterion according to the limit state combines two important problems: on the one hand we must refer to the resistance of the materials we will use for the foundation structure and on the other hand we must consider the double valence of the terrain, which integrates with the structure can take on a function as both resistive and stressful. To consider the above norms set out some typologies of boundary states: Limit state of Equilibrium as a rigid body (EQU), Limit state of GroundResistance (GEO) and Limit state of Structure Resistance (STR).

For ULS compliant controls the norms provide for two different design approaches defined as "Approch1" and "Approch2". In each approach there are different combinations of groups of partial coefficients for loads (A), for geotechnical parameters (M) and for global resistance (R).

"Approch2" which consists of only a combination of coefficients generally addresses less conservative results than "Approch1" which we have chosen to use in the object in question. According to this approach "Approch1" two combinations of coefficients are provided

Combination 1 (STR) : (A1 + M1 + R1)

Combination 2 (GEO) : (A2 + M2 + R2)

Combination (STR) deals with structural dimensioning and defines high limit state in determining the strength of foundation elements. By applying this combination we have load increases (by means of group A1 coefficients) and we have unchanged the global system and terrain resistances (by means of M1eR1 coefficients).

The combination (GEO) deals with the geotechnical dimensioning of the work and addresses a reduction of terrain and global resistances of the system (by means of the coefficients of the group M2e R2) leaving unchanged the loads (by means of the coefficients of the group A2). In the case of the object in question, which also has the presence of seismicity, the combination of seismic action with other loads is done by using the unit partial safety coefficients for loads, and with the coefficients (GEO) for geotechnical parameters and resistances.

11. STRUCTURE DESCRIPTION

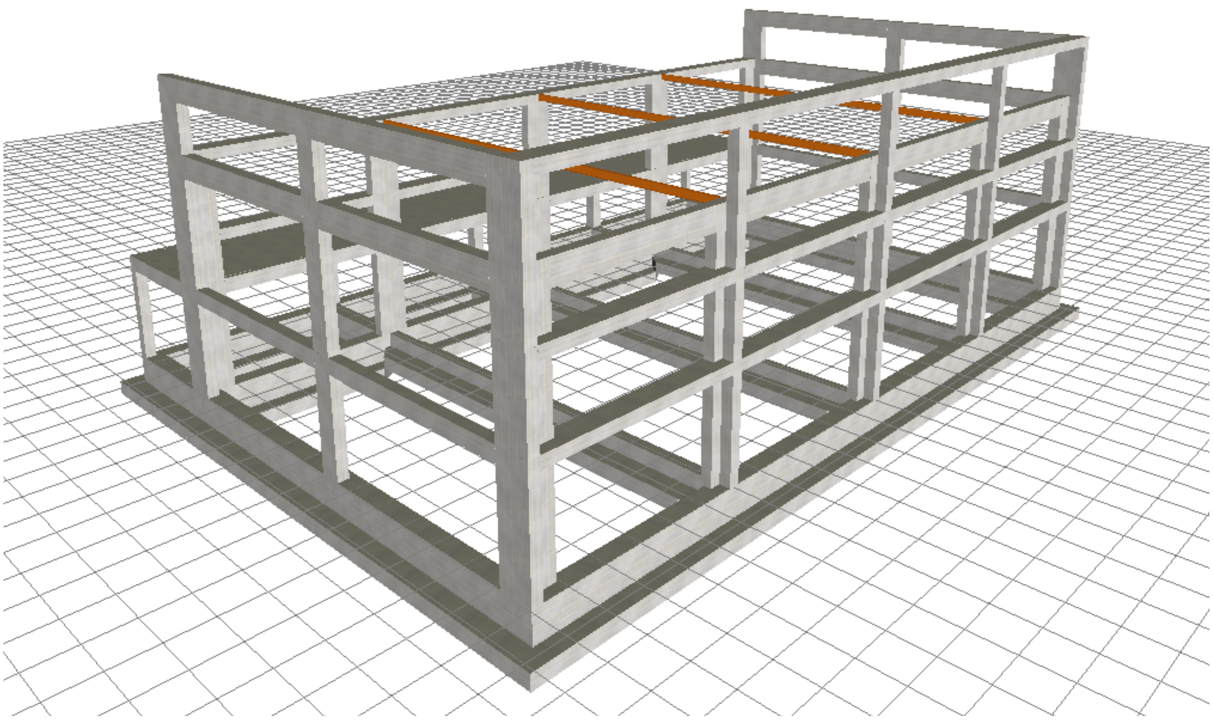
The building is designed with 1 floor above the ground, with two levels.

The heights of the two levels are as follows:

LEvel-1: 3.02 m

Level-2: 8.00 m

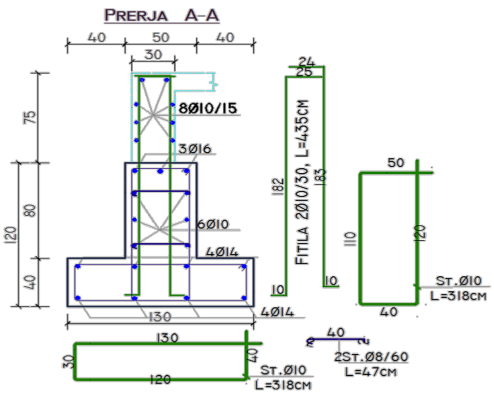
The object is conceived and calculated with frames giving priority to both directions of the object to guarantee the displacements allowed by the actions of external loads, mainly seismic ones.

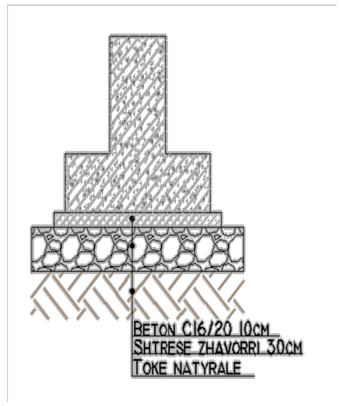


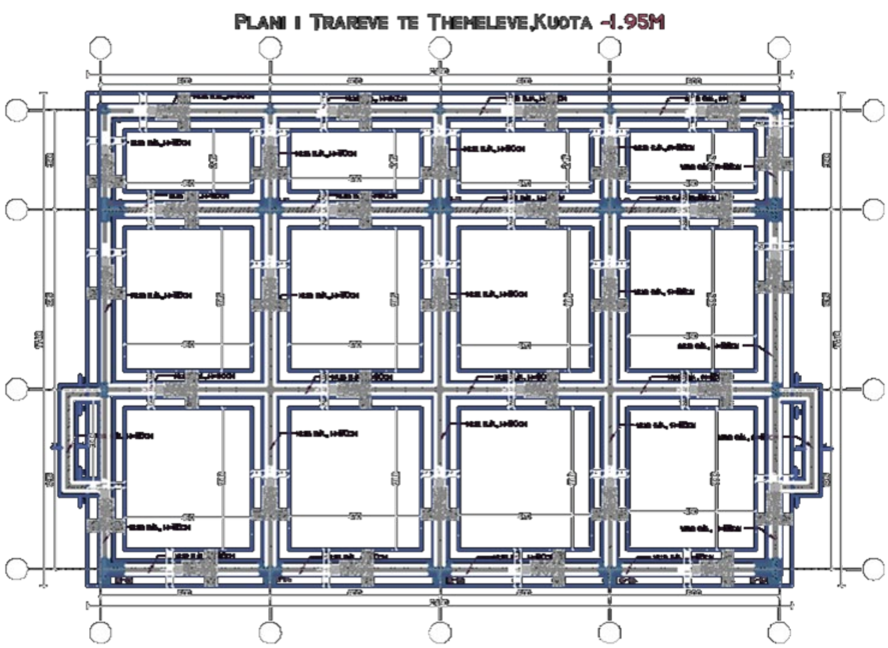
Themelet

The design of the foundations is based on the relevant geological report as well as the recommendations of this report. The foundations will be supported in the second layer, according to the study after this is reinforced with a 30 cm layer of compacted gravel. They are conceived with crossed beams with inverted T-section. The beams have a height of 120 cm, the depth needed to ensure the insertion of the building.

To reach the quota of 0.00 above the beam will be erected walls b / arms 30cm thick with a height of 75 cm. Under the foundation sole will be filled with a layer of concrete 16/20 with a thickness of 10cm, and a layer of gravel with a thickness of 30cm.

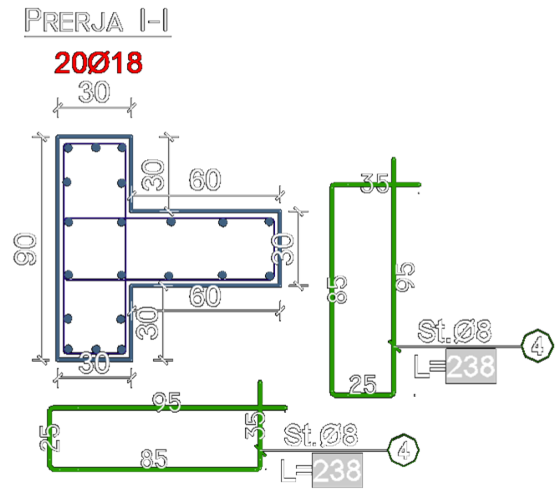






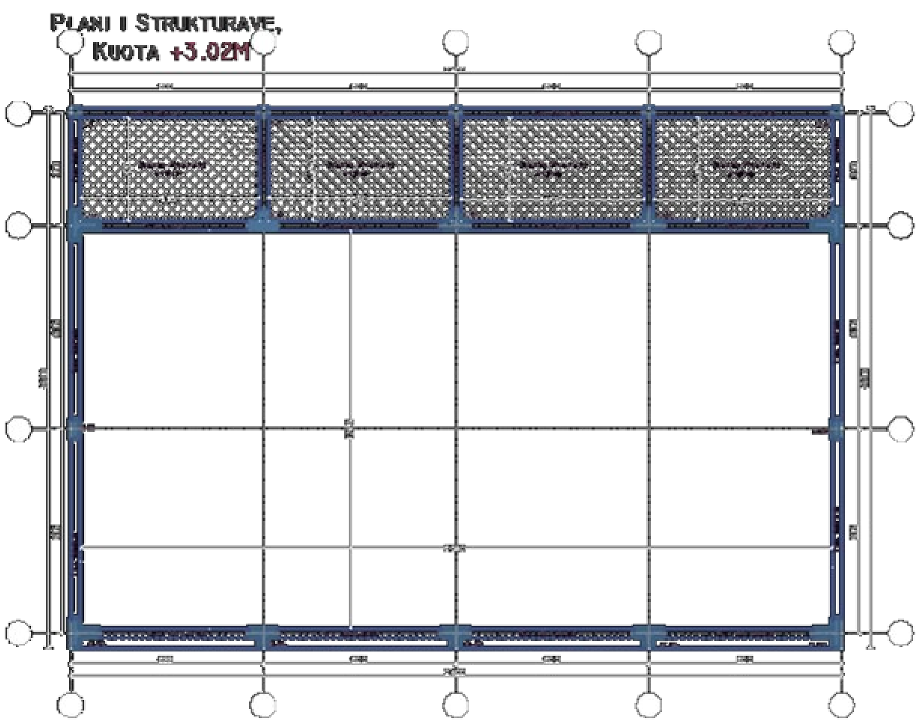
Kollonat

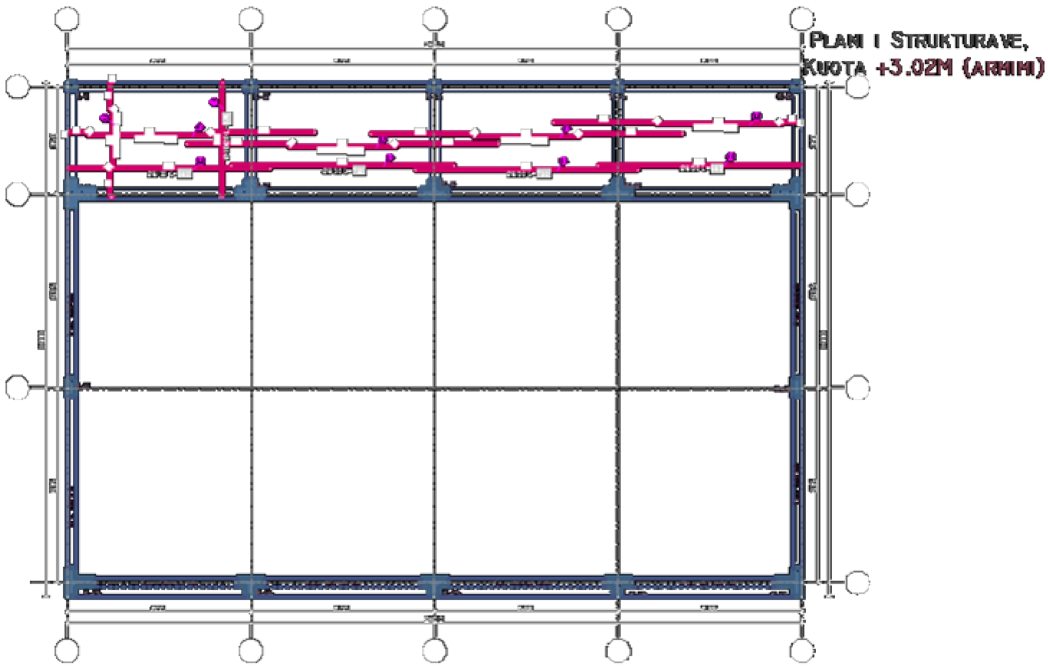
The columns have the shape of a rectangular cross section (bxh = 30x30cm) and (bxh = 30x60cm), and T section (bxh=30x90, 90x30cm) with unchanging section according to the height. The reinforcement will be made with bars ϕ18. The stirrups to be used will be ϕ8. The stirrups will be placed for the critical area every 10 cm, while for the non-critical area every 20 cm. Jointing of the column bars will be done at the level of the intersection slab, at two different levels.



Slabs

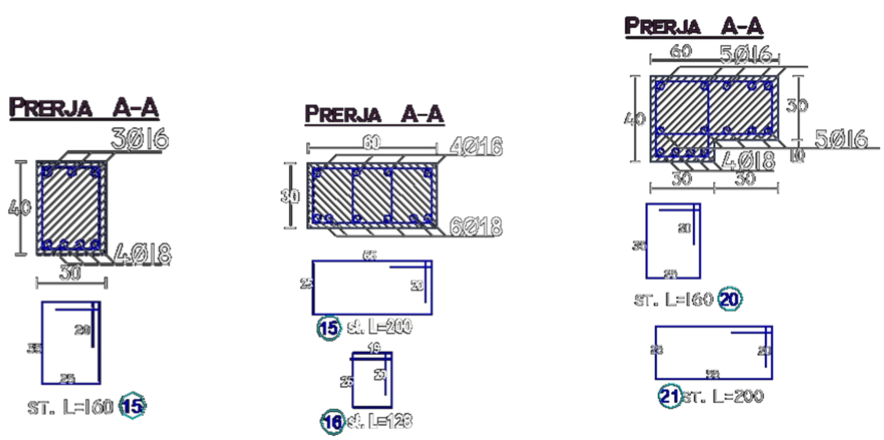
The horizontal structures are made of monolithic slabs, supported in two directions with thickness t = 17cm, and will be reinforced with ϕ8 every 20cm. The purpose of the selection is to better distribute the loads acting on it, through the beams of the building and to better release their role as a horizontal diaphragm.





Beams

The beams of the structure will have a rectangular cross section with dimensions bxh = 30x40cm, bxh = bxh = 60x30 cm and bea,s with L section bxh=60x40cm

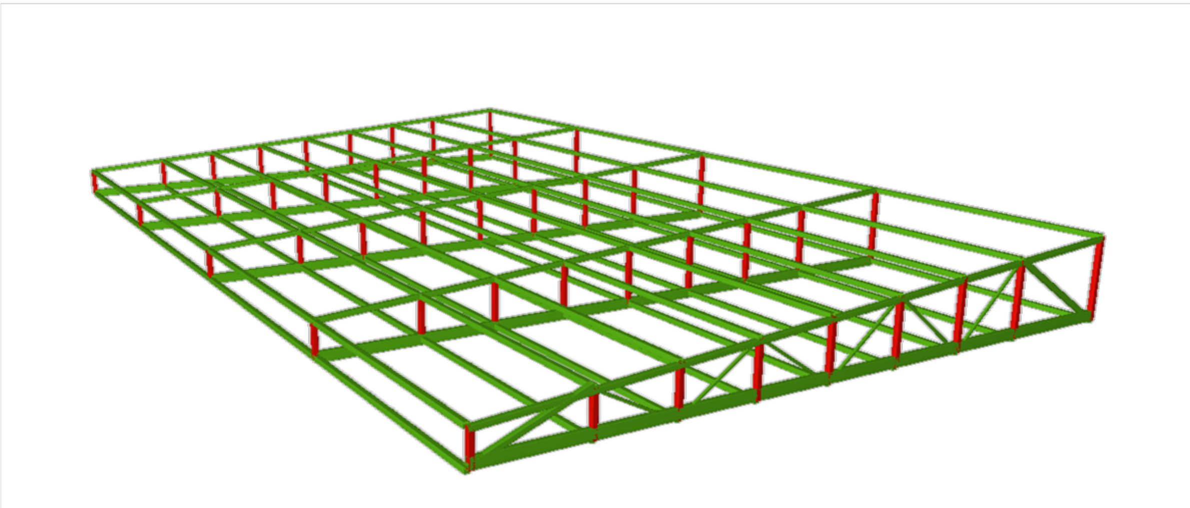


In the calculation of beams are placed trapezoidal or triangular loads coming from the soles as well as uniform loads coming from the walls.

The brick masonry in the building is provided with a thickness of 20 and 30 cm made with horizontal holes (lightened bricks). In the calculation scheme, the masonry load is accepted and evenly distributed in the floor with an intensity of 150 daN / m2. This allows the possibility of placing it in any place of the solet even if the layout of the premises is changed.

12. THE STEEL TRUSS

The cover of the Gym is made with steel truss with steel construction, and is covered with a sandwich panel according to the project drawings.



13. MATERIALS

► The steel provided is RHS profile 200x100x6 and RHA 100x100x6 and connections with RHS 50x50x6 Fe360

With these data :

E: Modulus of elasticity (MPa)	210000.00
μ: Poisson coefficients	0.30
G: Shear Modulus (MPa)	81000.00
fy: demolition limit (MPa)	235.00
α·t: Thermal swelling coefficient (m/m°C)	0.000012
γ: Weight (kN/m³)	77.01

14. COMPUTATIVE LOADS IN THE PROJECT

Load combinations will be determined according to the following criteria for the different project situation:

With combined coefficients

$$\sum_{j\geq 1}\gamma_{Gj}G_{kj}+\gamma_P P_k+\gamma_{Q1}\Psi_{p1}Q_{k1}+\sum_{i>1}\gamma_{Qi}\Psi_{ai}Q_{ki}$$

- Without combined coefficients

$$\sum_{j\geq 1}\gamma_{Gj}G_{kj}+\gamma_P P_k+\sum_{i\geq 1}\gamma_{Qi}Q_{ki}$$

G_k Permanent load
P_k Action before straining
Q_k Variable load
γ_G Partial safety factor for permanent loads
γ_P Partial safety factor for pre-stress actions
$\gamma_{Q,1}$ Partial safety factor for variable main load
$\gamma_{Q,i}$ Partial safety factor of the accompanying load
$\psi_{p,1}$ The main coefficient of variable load combination
$\psi_{a,i}$ Combined variable load combination coefficient

Ultimate Limit State U.L.S. Profiled Steel: Eurocode 3 and 4

Continuous or temporary				
	Partial safety factors (γ)		Combination coefficients (ψ)	
	Favorable	Unfavorable	primary (ψ_p)	companion (ψ_a)
Dead load (G)	1.000	1.350	-	-

Accidental fire				
	Partial safety factors (γ)		Combination coefficients (ψ)	
	Favorable	Unfavorable	primary (ψ_p)	companion (ψ_a)
Dead load (G)	1.000	1.000	-	-

Displacement

Variable loads without seismic loading		
	Partial safety factors (γ)	
	Favorable	Unfavorable
Dead load (G)	1.000	1.000

1. Coefficients for continuous or temporary situations

Comb.	SW
1	1.000
2	1.350

2. Coefficients for accidental fire situations

Comb.	SW
1	1.000

3. Displacement

Comb.	SW
1	1.000

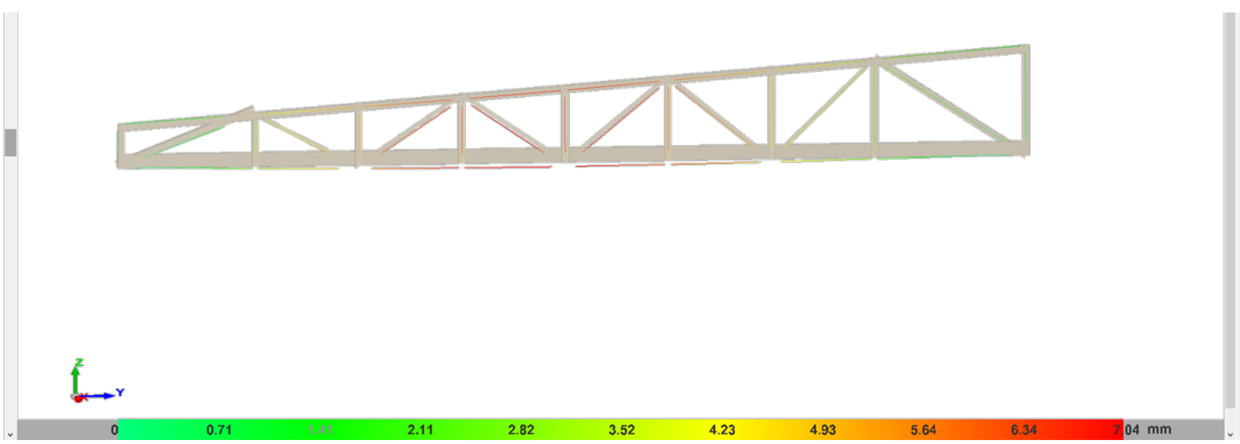
- Fire resistance

Code: EN1993-1-2: 2005: Design of steel structures - Part 1-2: General rules - design of structures in case of fire.

Resistance required: R 30

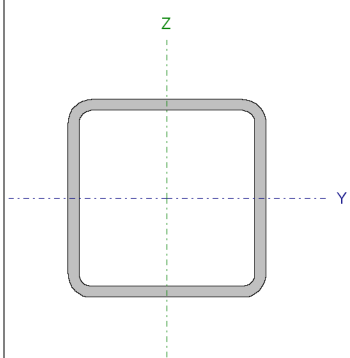
15 - Controls U.L.S. (FULL)

The controls are performed according to Eurocodes 3 and for illustration we give the basic calculations taking into account the most strained elements



Rod N18/N17 (and for all subsequent nodes)

Seksioni: RHS 100x100x6
Materiali: Steel (Fe360)



Nyjet		Gjatesia (m)	Karakteristikat mekanike			
Fillestare	Finale		Sip. (cm^2)	I_y[1] (cm4)	I_z[1] (cm4)	I_t[2] (cm4)
N18	N17	1.504	21.26	309.86	809.86	513.368

Shenime:
[1] Inercia në lidhje me aksin e treguar
[2] Momenti uniform torsional i inercisë

	Lidhja		Lidhje anesore	
	Plani XY	Plani XZ	Siper.	Poshte
β	1.00	1.00	0.00	0.00
L_K	1.504	1.504	0.000	0.000
C_m	1.000	1.000	1.000	1.000
C_1	-		1.000	

Notation:
β: Koeficienti i lidhjes
L_K: Gjatesia e lidhjes (m)
C_m: Koeficienti i momentit
C_1: Faktori i modifikimit ne momentin kritik

Situata e zjarrit

Rezistenca e kerkuar: R 30
Faktori i formes: 228.60 m-1
Temperatura maksimale e shufres: 832.0 °C

Bar	CHECKS (EUROCODE 3 EN 1993-1-1: 2005)													Status
	λ_w	N_t	N_c	M_Y	M_Z	V_Z	V_Y	M_YV_Z	M_ZV_Y	NM_YM_Z	$NM_YM_ZV_YV_Z$	M_t	M_tV_Z	M_tV_Y	
N18/N17	$\lambda_w \leq \lambda_{w,max}$ Verified	N_{Ed} = 0.00 D.N.P.[1]	x: 0.081 m η = 30.4	x: 1.45 m η = 7.0	x: 1.45 m η = 3.0	x: 0.081 m η = 0.7	x: 0.994 m η = 0.5	η < 0.1	η < 0.1	x: 1.45 m η = 37.7	η < 0.1	η = 2.1	x: 0.081 m η = 0.5	x: 0.081 m η = 0.3	**VERIFIED η = 37.7**

Notation:
λ_w: Crushing of the web induced by the compressed flange
N_t: Resistance to axial tension
N_c: Compression resistance
M_Y: Y - Axis bending resistance
M_Z: Z - Axis bending resistance
V_Z: Resistance to shear in the Z direction
V_Y: Resistance to shear in the Y direction
M_YV_Z: Combined bending moment Y and shear force Z resistance
M_ZV_Y: Combined bending moment Z and shear force Y resistance
NM_YM_Z: Combined bending and axial resistance
$NM_YM_ZV_YV_Z$: Combined bending, axial and shear resistance
M_t: Torsional resistance
M_tV_Z: Combined Z shear and torsional resistance
M_tV_Y: Combined Y shear and torsional resistance
x: Distance to the origin of the bar
η: Usage coefficient (%)
D.N.P.: Not applicable

Checks that do not proceed (D.N.P.):
[1] The check does not proceed, as there is no tensile axial force.

Crushing of the web induced by the compressed flange- Outside temperature (Eurocode 3 EN 1993-1-5: 2006, Article 8)

The following criteria must be satisfied:

$$\frac{\mathbf{h_w}}{\mathbf{t_w}} \leq k \frac{E}{f_{yf}} \sqrt{\frac{A_w}{A_{fc,ef}}} \qquad 26.15 \leq 199.27 \checkmark$$

h_w: Height of the web.	h_w	:	*170.00*	mm
t_w: Web thickness.	t_w	:	*6.50*	mm
A_w: Area of the web.	A_w	:	*11.05*	cm²
$A_{fc,ef}$: Reduced area of the compressed flange.	$A_{fc,ef}$	:	*20.00*	cm²
k: Coefficient which depends on the class of the section.	k	:	*0.30*	
E: Modulus of elasticity.	E	:	*210000*	MPa
f_{yf}: Steel elastic limit of the compressed flange.	f_{yf}	:	*235.00*	MPa

$$f_{yf} = f_y$$

Resistance to axial tension- Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.3)

The check does not proceed, as there is no tensile axial force.

Compression resistance - Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.4)

The following criteria must be satisfied:

$$\eta = \frac{N_{c,Ed}}{N_{c,Rd}} \leq 1$$

η : *0.009* ✓

The most unfavorable case for the design forces at node N1, occurs for the load combination 1.35 · SW.

$N_{c,Ed}$: The most unfavorable case of the designed axial compressive force. — $N_{c,Ed}$: *11.56* kN

bearing capacity calculation value against normal force Nc, Rd should be taken:

$$\mathbf{N_{c,Rd}} = A \cdot f_{yd}$$

$N_{c,Rd}$: *1264.30* kN

Class: Section Class, depending on its deformation capacity and the development of plastic resistance of the compressed elements of a section.	Type	:	*1*	
A:Total section area for type 1, 2 and 3.	A	:	*53.80*	cm²
f_{yd}: Steel design strength.	f_{yd}	:	*235.00*	MPa

$$\mathbf{f_{yd}} = f_y / \gamma_{M0}$$

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1)	f_y	:	*235.00*	MPa
γ_{M0}: Partial safety factor of the material.	γ_{M0}	:	*1.00*	

Buckling resistance: (Eurocode 3 EN 1993-1-1: 2005, Article 6.3.1)
If the slenderness $\bar{\lambda} \leq 0.2$ or the ratio $\mathbf{N_{c,Ed} / N_{cr}} \leq 0.04$, the buckling effect can be ignored and only the resistance of the transverse section has to be checked.

$\bar{\lambda}$: Reduced slenderness. $\bar{\lambda}$: *0.57*

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}}$$

$N_{c,Ed}/N_{cr}$: Axis force ratio. $N_{c,Ed}/N_{cr}$: *0.003*

Where:
A: Area of the gross section for class 1, 2 and 3 sections.
fy: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1)
Ncr: Critical elastic buckling axial force, obtained from the smallest of the following values:
Ncr,z: Critical elastic buckling axial force with respect to the Z axis.
Ncr,y: Critical elastic buckling axial force with respect to the Y axis.

A	:	*53.80*	cm²
f_y	:	*235.00*	MPa
N_{cr}	:	*3943.06*	kN
$N_{cr,y}$	:	*10896.55*	kN
$N_{cr,z}$	:	*3943.06*	kN
$N_{cr,T}$	:	∞	

The following criteria must be satisfied:

$$\eta = \frac{M_{Ed}}{M_{c,Rd}} \leq 1$$

η : ***0.005*** ✓

For positive bending:

The worst case design force occurs at a point situated at node N3, for load combination 1.35SW

M_{Ed}^+: Worst case design bending moment. **M_{Ed}^+** : *0.54* kN·m

For negative bending:

The worst case design force occurs at a point situated at node N3, for load combination 1.35SW-

M_{Ed}^- : *0.00* kN·m

M_{Ed}^-: Worst case design bending moment.

$$\mathbf{M_{c,Rd}} = W_{pl,y} \cdot f_{yd}$$

$M_{c,Rd}$: *100.93* kN·m

The design bending moment resistance **$M_{c,Rd}$** is given by:

Class: The class of the section, depending on its deformation ability and the development of the plastic resistance of the flat elements of a section presented in simple bending. **Class** : *1*

Wpl, y: Plastic strength modulus corresponding to the fibre with greatest tension, for class 1 and 2 sections. **$W_{pl,y}$** : *429.50* cm³

f_{yd}: Steel design strength. **f_{yd}** : *235.00* MPa

$$\mathbf{f_{yd}} = f_y / \gamma_{M0}$$

$\mathbf{f_y}$: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) $\mathbf{f_y}$: *235.00* MPa

$\gamma_{\mathbf{M0}}$: Partial safety factor for the material $\gamma_{\mathbf{M0}}$: *1.00*

Lateral resistance of connections: (Eurocode 3 EN 1993-1-1: 2005, Article 6.3.2)

It does not continue, since the side link lengths are invalid.

Z - Axis bending resistance - Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.5)

The following criteria must be satisfed:

$$\eta = \frac{M_{Ed}}{M_{c,Rd}} \leq 1$$

η : ***0.001*** ✓

For positive bending:

$\mathbf{M_{Ed}^+}$: *0.00* kN·m

$\mathbf{M_{Ed}^+}$: Worst case design bending moment.

For negative bending:

The worst case design force occurs at a point situated at node N3, for load combination 1.35SW $\mathbf{M_{Ed}^-}$: *0.06* kN·m

$\mathbf{M_{Ed}^-}$: Worst case design bending moment.

$$\mathbf{M_{c,Rd}} = W_{pl,y} \cdot f_{yd}$$

$\mathbf{M_{c,Rd}}$: *47.89* kN·m

For positive bending:

Where:

Class: The class of the section, depending on its deformation ability and the development of the plastic resistance of the flat elements of a section presented in simple bending. **Class** : *1*

Wpl, y: Plastic strength modulus corresponding to the fibre with greatest tension, for class 1 and 2 sections. $\mathbf{W_{pl,z}}$: *203.80* cm³

$\mathbf{f_{yd}}$: Steel design strength. $\mathbf{f_{yd}}$: *235.00* MPa

$$\mathbf{f_{yd}} = f_y / \gamma_{M0}$$

$\mathbf{f_y}$: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) $\mathbf{f_y}$: *235.00* MPa

$\gamma_{\mathbf{M0}}$: Partial safety factor for the material $\gamma_{\mathbf{M0}}$: *1.00*

Resistance to shear in the Z direction - Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.6)

Resistance to shear in the Y direction - Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.6)

The following criteria must be satisfed:

$$\eta = \frac{V_{Ed}}{V_{c,Rd}} \leq 1$$

$\eta <$ ***0.001*** ✓

The most unfavorable case of force design for load combination 1.35 · SW.

V_{Ed}: Worst case design shear force. **V_{Ed}** : *0.03* kN

The shear resistance **$V_{c,Rd}$** is given by:

$$\mathbf{V_{c,Rd}} = A_V \cdot \frac{f_{yd}}{\sqrt{3}}$$

$V_{c,Rd}$: *580.02* kN

where:

A_v: Transverse shear area. **A_v** : *42.75* cm²

$$\mathbf{A_v} = A - d \cdot t_w$$

where:

A: Area of the gross section. **A** : *53.80* cm²

d Height of the web. **d** : *170.00* mm

t_w: Web thickness.. **t_w** : *6.50* mm

f_{yd}: Steel design strength. **f_{yd}** : *235.00* MPa

$$\mathbf{f_{yd}} = f_y / \gamma_{M0}$$

where:

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) **f_y** : *235.00* MPa

γ_{M0}: Partial safety factor of the material. **γ_{M0}** : *1.00*

The following criteria must be satisfed:

$$\eta = \frac{V_{Ed}}{V_{c,Rd}} \leq 1$$

η : *0.001* ✓

The most unfavorable case of force design for load combination 1.35 · SW.

V_{Ed}: Worst case design shear force. V_{Ed} : *0.30* kN

The shear resistance **$V_{c,Rd}$** is given by:

$$\mathbf{V_{c,Rd}} = A_V \cdot \frac{f_{yd}}{\sqrt{3}}$$

$V_{c,Rd}$: *244.90* kN

Where:

A_v: Transverse shear area.	A_v :	*18.05*	cm²
Where:			
h: length of section	h :	*190.00*	mm
t_w: Web thickness..	t_w :	*6.50*	mm
f_{yd}:reistenca llogaritese.	f_{yd} :	*235.00*	MPa

$$\mathbf{f_{yd}} = f_y / \gamma_{M0}$$

where:

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1)	f_y :	*235.00*	MPa
γ_{M0}: Partial safety factor of the material.	γ_{M0} :	*1.00*	

Shear buckling of the web: (Eurocode 3 EN 1993-1-5: 2006, Article5)
Even though transverse stiffeners have not been provided, it is not necessary to check the buckling resistance of the web, as the following is verified:

$$\frac{\mathbf{d}}{\mathbf{t_w}} < \frac{72}{\eta} \cdot \varepsilon$$

20.62 < 60.00 ✓

where:

λ_w: Slenderness of the web.	λ_w :	*20.62*

$$\lambda_{\mathbf{w}} = \frac{d}{t_w}$$

λ_{max}: Maximum slenderness.	λ_{max} :	*60.00*

$$\lambda_{\mathbf{max}} = \frac{72}{\eta} \cdot \varepsilon$$

η: Coefficient which allows to consider the additional resistance in plastic regime because of hardening due to deformed material.	η :	*1.20*
ε: Reduction factor.	ε :	*1.00*

$$\varepsilon = \sqrt{\frac{f_{ref}}{f_y}}$$

where:

f_{ref}: Reference elastic limit.	f_{ref} :	*235.00*	MPa
f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1)	f_y :	*235.00*	MPa

Combined bending moment Y and shear force Z resistance- Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.8)

It is not necessary to reduce the design bending resistance, as the worst case shear force **V_{Ed}** is not greater than 50% of the design shear resistance **$V_{c,Rd}$**.

$$\mathbf{V_{Ed}} \le \frac{\mathbf{V_{c,Rd}}}{2}$$

0.30 kN ≤ **122.45 kN** ✓

The worst case design forces occur for load combination 1.35SW-

V_{Ed}: Worst case design shear force. **V_{Ed}** : *0.30* kN

$\mathbf{V_{c,Rd}}$: Design resistant shear force. $\mathbf{V_{c,Rd}}$: *244.90* kN

Combined bending moment Z and shear force Y resistance - Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.8)

It is not necessary to reduce the design bending resistance, as the worst case shear force $\mathbf{V_{Ed}}$ is not greater than 50% of the design shear resistance $\mathbf{V_{c,Rd}}$.

$$\mathbf{V_{Ed}} \le \frac{\mathbf{V_{c,Rd}}}{2}$$

0.03 kN ≤ 290.01 kN ✓

The worst case design forces occurs at node N1 for load combination 1.35SW

$\mathbf{V_{Ed}}$: Worst case design shear force. $\mathbf{V_{Ed}}$: *0.03* kN

$\mathbf{V_{c,Rd}}$: Design resistant shear force. $\mathbf{V_{c,Rd}}$: *580.02* kN

Combined bending and axial resistance - Outside temperature(Eurocode 3 EN 1993-1-1: 2005, Article 6.2.9)

The following criteria must be satisfied:

$$\eta = \left[\frac{M_{y,Ed}}{M_{N,Rd,y}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,Rd,z}}\right]^{\beta} \le 1$$

η : ***0.001*** ✓

$$\eta = \frac{N_{c,Ed}}{\chi_y \cdot A \cdot f_{yd}} + k_{yy} \cdot \frac{M_{y,Ed}}{\chi_{LT} \cdot W_{pl,y} \cdot f_{yd}} + k_{yz} \cdot \frac{M_{z,Ed}}{W_{pl,z} \cdot f_{yd}} \le 1$$

η : ***0.014*** ✓

$$\eta = \frac{N_{c,Ed}}{\chi_z \cdot A \cdot f_{yd}} + k_{zy} \cdot \frac{M_{y,Ed}}{\chi_{LT} \cdot W_{pl,y} \cdot f_{yd}} + k_{zz} \cdot \frac{M_{z,Ed}}{W_{pl,z} \cdot f_{yd}} \le 1$$

η : ***0.012*** ✓

The worst case design forces occurs at node N3 for load combination 1.35SW

Where:

$\mathbf{N_{c,Ed}}$: Compressive axial force to be withstood from the analysis. $\mathbf{N_{c,Ed}}$: *10.08* kN

$\mathbf{M_{y,Ed}}$, $\mathbf{M_{z,Ed}}$: Worst case bending moments, in accordance with the Y and Z axes, respectively. $\mathbf{M_{y,Ed}^{+}}$: *0.54* kN·m

$\mathbf{M_{z,Ed}^{-}}$: *0.06* kN·m

Class: Section class, according to its deformation capacity and plastic resistance development of its flat elements, for axial load and simple bending. **Class** : *1*

$\mathbf{M_{N,Rd,y}}$, $\mathbf{M_{N,Rd,z}}$: Reduced design plastic resistant bending moments, about $\mathbf{M_{N,Rd,y}}$: *100.93* kN·m the Y and Z axes, respectively. **$M_{N,Rd,z}$** : *47.89* kN·m

$$\mathbf{M_{N,Rd,y}} = M_{pl,Rdy} \cdot (1-n)/(1-0.5\cdot a) \le M_{pl,Rd,y}$$

$$n \le a \rightarrow \mathbf{M_{N,Rd,z}} = M_{pl,Rd,z}$$

$$\alpha = 2 \;;\; \beta = 5\cdot n \ge 1$$

α : *2.000*

β : *1.000*

Where:

$$\mathbf{n} = N_{c,Ed}/N_{pl,Rd}$$

n : *0.008*

$N_{pl,Rd}$: Compressive resistance of the gross section. **$N_{pl,Rd}$** : *1264.30* kN

$M_{pl,Rd,y}$, $M_{pl,Rd,z}$: Bending resistance of the gross section in plastic conditions, with respect to the Y and Z axes, respectively.

$M_{pl,Rd,y}$: *100.93* kN·m

$M_{pl,Rd,z}$: *47.89* kN·m

$$\mathbf{a} = (A - 2\cdot b\cdot t_f)/A \le 0.5$$

a : *0.26*

A: Area of the gross section. **A** : *53.80* cm²

b: Flange width. **b** : *20.00* cm

t_f: Thickness of the flange. **t_f** : *10.00* mm

Buckling resistance: (Eurocode 3 EN 1993-1-1: 2005, Article 6.3.3)

A: Area of the gross section. **A** : *53.80* cm²

$W_{pl,y}$, $W_{pl,z}$: Plastic resistance moduli corresponding to the fibre with greatest stress about the Y and Z axes, respectively.

$W_{pl,y}$: *429.50* cm³

$W_{pl,z}$: *203.80* cm³

f_{yd}: Steel design strength. **f_{yd}** : *235.00* MPa

$$\mathbf{f_{yd}} = f_y/\gamma_{M1}$$

where:

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) **f_y** : *235.00* MPa

γ_{M1}: Partial safety factor of the material **γ_{M1}** : *1.00*

K_{yy}, K_{yz}, K_{zy}, K_{zz}: Interaction coefficients.

$$\mathbf{k_{yy}} = C_{m,y}\cdot C_{m,LT}\cdot \frac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,y}}}\cdot\frac{1}{C_{yy}}$$

K_{yy} : *1.00*

$$\mathbf{k_{yz}} = C_{m,z}\cdot \frac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}}\cdot\frac{1}{C_{yz}}\cdot 0.6\cdot\sqrt{\frac{w_z}{w_y}}$$

K_{yz} : *0.70*

$$\mathbf{k_{zy}} = C_{m,y}\cdot C_{m,LT}\cdot \frac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,y}}}\cdot\frac{1}{C_{zy}}\cdot 0.6\cdot\sqrt{\frac{w_y}{w_z}}$$

K_{zy} : *0.52*

$$\mathbf{k_{zz}} = C_{m,z}\cdot \frac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,z}}}\cdot\frac{1}{C_{zz}}$$

K_{zz} : *1.00*

Auxiliary terms:

$$\mu_y = \frac{1 - \frac{N_{Ed}}{N_{cr,y}}}{1 - \chi_y \cdot \frac{N_{Ed}}{N_{cr,y}}}$$

μ_y : 1.00

$$\mu_z = \frac{1 - \frac{N_{Ed}}{N_{cr,z}}}{1 - \chi_z \cdot \frac{N_{Ed}}{N_{cr,z}}}$$

μ_z : 1.00

$$C_{yy} = 1 + (w_y - 1) \cdot \left[\left(2 - \frac{1.6}{w_y} \cdot C_{my}^2 \cdot \bar{\lambda}_{max} - \frac{1.6}{w_y} \cdot C_{my}^2 \cdot \bar{\lambda}_{max}^2\right) \cdot n_{pl} - b_{LT}\right] \geq \frac{W_{el,y}}{W_{pl,y}}$$

C_{yy} : 1.00

$$C_{yz} = 1 + (w_z - 1) \cdot \left[\left(2 - 14 \cdot \frac{C_{mz}^2 \cdot \bar{\lambda}_{max}^2}{w_z^5}\right) \cdot n_{pl} - c_{LT}\right] \geq 0.6 \cdot \sqrt{\frac{w_z}{w_y}} \cdot \frac{W_{el,z}}{W_{pl,z}}$$

C_{yz} : 1.01

$$C_{zy} = 1 + (w_y - 1) \cdot \left[\left(2 - 14 \cdot \frac{C_{my}^2 \cdot \bar{\lambda}_{max}^2}{w_y^5}\right) \cdot n_{pl} - d_{LT}\right] \geq 0.6 \cdot \sqrt{\frac{w_y}{w_z}} \cdot \frac{W_{el,y}}{W_{pl,y}}$$

C_{zy} : 1.00

$$C_{zz} = 1 + (w_z - 1) \cdot \left[\left(2 - \frac{1.6}{w_z} \cdot C_{mz}^2 \cdot \bar{\lambda}_{max} - \frac{1.6}{w_z} \cdot C_{mz}^2 \cdot \bar{\lambda}_{max}^2 - e_{LT}\right) \cdot n_{pl}\right] \geq \frac{W_{el,z}}{W_{pl,z}}$$

C_{zz} : 1.00

$$a_{LT} = 1 - \frac{I_t}{I_y} \geq 0$$

a_{LT} : 0.99

$$b_{LT} = 0.5 \cdot a_{LT} \cdot \bar{\lambda}_0^2 \cdot \frac{M_{y,Ed}}{\chi_{LT} \cdot M_{pl,Rd,y}} \cdot \frac{M_{z,Ed}}{M_{pl,Rd,z}}$$

b_{LT} : 0.00

$$c_{LT} = 10 \cdot a_{LT} \cdot \frac{\bar{\lambda}_0^2}{5 + \bar{\lambda}_z^4} \cdot \frac{M_{y,Ed}}{C_{m,y} \cdot \chi_{LT} \cdot M_{pl,Rd,y}}$$

c_{LT} : 0.00

$$d_{LT} = 2 \cdot a_{LT} \cdot \frac{\bar{\lambda}_0}{0.1 + \bar{\lambda}_z^4} \cdot \frac{M_{y,Ed}}{C_{m,y} \cdot \chi_{LT} \cdot M_{pl,Rd,y}} \cdot \frac{M_{z,Ed}}{C_{m,z} \cdot M_{pl,Rd,z}}$$

d_{LT} : 0.00

$$e_{LT} = 1.7 \cdot a_{LT} \cdot \frac{\bar{\lambda}_0}{0.1 + \bar{\lambda}_z^4} \cdot \frac{M_{y,Ed}}{C_{m,y} \cdot \chi_{LT} \cdot M_{pl,Rd,y}}$$

e_{LT} : 0.00

$$w_y = \frac{W_{pl,y}}{W_{el,y}} \leq 1.5$$

w_y : 1.11

$$w_z = \frac{W_{pl,z}}{W_{el,z}} \leq 1.5$$

w_z : 1.50

$$n_{pl} = \frac{N_{Ed}}{N_{pl,Rd}}$$

n_{pl} : *0.01*

Given that:

$$\bar{\lambda}_0 \leq 0.2 \cdot \sqrt{C_1} \cdot \sqrt[4]{\left(1-\frac{N_{Ed}}{N_{cr,z}}\right)\cdot\left(1-\frac{N_{Ed}}{N_{cr,T}}\right)}$$

0.00 ≤ **0.20**

$$C_{m,y} = C_{m,y,0}$$

$C_{m,y}$: *1.00*

$$C_{m,z} = C_{m,z,0}$$

$C_{m,z}$: *1.00*

$$C_{m,LT} = 1.00$$

$C_{m,LT}$: *1.00*

$C_{m,y,0}$, $C_{m,z,0}$: Equivalent uniform bending moment factors.	**$C_{m,y,0}$** : *1.00*
	$C_{m,z,0}$: *1.00*
C_1: Factor which depends on the support conditions and bending moment envelope of the bar.	**C_1** : *1.00*
χ_y, χ_z: Buckling reduction coefficients, about the Y and Z axes, respectively.	**χ_y** : *1.00*
	χ_z : *1.00*
χ_{LT}: Lateral buckling reduction coefficient.	**χ_{LT}** : *1.00*
$\bar{\lambda}_{max}$: Maximum slenderness between $\bar{\lambda}_y$ and $\bar{\lambda}_z$.	**$\bar{\lambda}_{max}$** : *0.57*
$\bar{\lambda}_y$, $\bar{\lambda}_z$: Reduced slendernesses with respect to the Y and Z axes, respectively.	**$\bar{\lambda}_y$** : *0.34*
	$\bar{\lambda}_z$: *0.57*
$\bar{\lambda}_{LT}$: Reduced slenderness.	**$\bar{\lambda}_{LT}$** : *0.00*
$\bar{\lambda}_0$: Reduced slenderness, with respect to lateral buckling, for a uniform bending moment.	**$\bar{\lambda}_0$** : *0.00*
$W_{el,y}$, $W_{el,z}$: Elastic resistant modules corresponding to the compressed fibre, about the Y and Z axes, respectively.	**$W_{el,y}$** : *388.63* cm^3
	$W_{el,z}$: *133.60* cm^3
$N_{cr,y}$: Critical elastic buckling axial force with respect to the Y axis.	**$N_{cr,y}$** : *10896.55* kN
$N_{cr,z}$: Critical elastic buckling axial force with respect to the Z axis.	**$N_{cr,z}$** : *3943.06* kN
$N_{cr,T}$: Axial force of critical elastic bond due to torsion.	**$N_{cr,T}$** : ∞
I_y: Moment of inertia of the gross section, with respect to the Y-axis.	**I_y** : *3692.00* cm4
I_t: Uniform torsional moment of inertia.	**I_t** : *20.98* cm4

<u>Combined bending, axial and shear resistance – Outside temperature</u>(Eurocode 3 EN 1993-1-1: 2005, Article 6.2.10)

It is not necessary to reduce the design bending and axial force resistance, as the buckling effect can be ignored due to shear. Additionally, the worst case design shear force **V_{Ed}** is less than or equal to 50% of the design shear resistance **$V_{c,Rd}$**.

The worst case design forces occur for load combination SW-0.3·SX-SY.

$$V_{Ed,y} \leq \frac{V_{c,Rd,y}}{2}$$

0.03 kN ≤ 290.01 kN ✓

where:

$V_{Ed,y}$: Worst case design shear force.	$V_{Ed,y}$	:	*0.03*	kN
$V_{c,Rd,y}$: Design resistant shear force.	$V_{c,Rd,y}$	:	*580.02*	kN

Torsional resistance – Outside temperature(Eurocode 3 EN 1993-1-1: 2005, Article 6.2.7)

The control does not continue, as there is no torsional moment.

Combined Z shear and torsional resistance - Outside temperature(Eurocode 3 EN 1993-1-1: 2005, Article 6.2.7)

There is no interaction between the torsional and the shear force for any combination. Therefore the control is not done

Combined Y shear and torsional resistance - Outside temperature(Eurocode 3 EN 1993-1-1: 2005, Article 6.2.7)

There is no interaction between the torsional and the shear force for any combination. Therefore the control is not done

Axial tensile strength – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.3, and EN 1993-1-2: 2005, Article 4)

Control is not done, as there is no axial tensile force.

Compressive strength – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.4, and EN 1993-1-2: 2005, Article 4)

The following criteria must be satisfied:

$$\eta = \frac{N_{c,Ed}}{N_{c,Rd}} \leq 1$$

η : *0.038* ✓

$$\eta = \frac{N_{c,Ed}}{N_{b,Rd}} \leq 1$$

η : *0.058* ✓

The worst case design force occurs at node N1, for load combination SW

$N_{c,Ed}$: Worst case design compressive axial force.	$N_{c,Ed}$	:	*8.56*	kN

The normal design compression force $\mathbf{N_{c,Rd}}$ should be taken as::

$$\mathbf{N_{c,Rd}} = A \cdot f_{yd}$$

$N_{c,Rd}$: *222.45* kN

Ku:

Class: Section class, depending on its deformation capacity and development of plastic resistance of the compressed elements of a section.	Class	:	*2*	
A: Area of the gross section for class 1, 2 and 3 sections.	A	:	*53.80*	cm²
$\mathbf{f_{yd}}$: Steel design strength.	f_{yd}	:	*41.35*	MPa

$$\mathbf{f_{yd}} = f_{y,\theta} / \gamma_{M,\theta}$$

where:

$f_{y,\theta}$: Reduced elastic limit for the temperature reached by the section. $f_{y,\theta}$: *41.35* MPa

$$\mathbf{f_{y,\theta}} = f_y \cdot k_{y,\theta}$$

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) f_y : *235.00* MPa

$k_{y,\theta}$: Elastic limit reduction factor for the temperature reached by the section. $k_{y,\theta}$: *0.18*

$\gamma_{M,\theta}$: Partial material safety factor. $\gamma_{M,\theta}$: *1.00*

Buckling resistance: (Eurocode 3 EN 1993-1-1: 2005, Article 6.3.1)
The calculated connection resistance Nb, Rd of a compression rod is given by:

$$\mathbf{N_{b,Rd}} = \chi \cdot A \cdot f_{yd}$$

$N_{b,Rd}$: *147.53* kN

where:

A: Area of the gross section for class 1, 2 and 3 sections A : *53.80* cm²

f_{yd}: Steel design strength. f_{yd} : *41.35* MPa

$$\mathbf{f_{yd}} = f_{y,\theta} / \gamma_{M,\theta}$$

where:

$f_{y,\theta}$: Reduced elastic limit for the temperature reached by the section. $f_{y,\theta}$: *41.35* MPa

$$\mathbf{f_{y,\theta}} = f_y \cdot k_{y,\theta}$$

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) f_y : *235.00* MPa

$k_{y,\theta}$: Elastic limit reduction factor for the temperature reached by the section. $k_{y,\theta}$: *0.18*

$\gamma_{M,\theta}$: Partial material safety factor $\gamma_{M,\theta}$: *1.00*

χ: Buckling reduction coefficients.

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \left(\bar{\lambda}\right)^2}} \leq 1$$

χ_y : *0.79*
χ_z : *0.66*

where:

$$\Phi = 0.5 \cdot \left[1 + \alpha \cdot \bar{\lambda} + \bar{\lambda}^2\right]$$

ϕ_y : *0.68*
ϕ_z : *0.87*

α: Elastic imperfection coefficient α_y : *0.65*
α_z : *0.65*

$\bar{\lambda}$: Reduced slenderness..

$$\bar{\lambda} = k_{\lambda,\theta} \cdot \sqrt{\frac{A \cdot f_y}{N_{cr}}}$$

$\bar{\lambda}_y$: *0.36*
$\bar{\lambda}_z$: *0.60*

$k_{\lambda,\theta}$: Reduced detail of increase of slenderness for temperature reached by section. $k_{\lambda,\theta}$: *1.05*

N_{cr}: Critical elastic buckling axial force, obtained from the smallest of the following values: N_{cr} : *3943.06* kN

$N_{cr,y}$: Critical elastic buckling axial force with respect to the Y axis. $N_{cr,y}$: *10896.55* kN

$N_{cr,z}$: Critical elastic buckling axial force with respect to the Z axis $N_{cr,z}$: *3943.06* kN

$N_{cr,T}$: Critical elastic buckling axial force due to torsion. $N_{cr,T}$: ∞

Y – Axis bending resistance – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.5, and EN 1993-1-2: 2005, Article 4)

The following criteria must be satisfied:

$$\eta = \frac{M_{Ed}}{M_{c,Rd}} \leq 1$$

η : ***0.022*** ✓

For positive bending:

The worst case design force occurs at a point situated at node N3, for load combination SW

M_{Ed}^{+}: Worst case design bending moment.. — M_{Ed}^{+} : *0.40* kN·m

For negative bending:

M_{Ed}^{-}: Worst case design bending moment. — M_{Ed}^{-} : *0.00* kN·m

The design bending moment resistance **$M_{c,Rd}$** is given by:

$$\mathbf{M_{c,Rd}} = W_{pl,y} \cdot f_{yd}$$

$M_{c,Rd}$: *17.76* kN·m

Where:

Class: Section class, depending on its deformation capacity and development of plastic resistance of the flat elements of a section submitted to simple bending. — **Class** : *2*

$W_{pl,y}$: Plastic strength modulus corresponding to the fibre with greatest tension, for class 1 and 2 sections. — $W_{pl,y}$: *429.50* cm³

f_{yd}: Steel design strength. — f_{yd} : *41.35* MPa

$$\mathbf{f_{yd}} = f_{y,\theta} / \gamma_{M,\theta}$$

Where:

$f_{y,\theta}$: Reduced elastic limit for the temperature reached by the section. — $f_{y,\theta}$: *41.35* MPa

$$\mathbf{f_{y,\theta}} = f_y \cdot k_{y,\theta}$$

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) — f_y : *235.00* MPa

$k_{y,\theta}$: Elastic limit reduction factor for the temperature reached by the section. — $k_{y,\theta}$: *0.18*

$\gamma_{M,\theta}$: Partial material safety factor — $\gamma_{M,\theta}$: *1.00*

Lateral connection resistance: (Eurocode 3 EN 1993-1-1: 2005, Article 6.3.2)

It does not continue, since the side link lengths are invalid.

Z – Axis bending resistance – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.5, and EN 1993-1-2: 2005, Article 4)

The following criteria must be satisfied:

$$\eta = \frac{M_{Ed}}{M_{c,Rd}} \leq 1$$

η : ***0.005*** ✓

For positive bending:

M_{Ed}^+: Worst case design bending moment. M_{Ed}^+ : *0.00* kN·m

For negative bending:

The worst case design force occurs at a point situated at node N3, for load combination SW

M_{Ed}^-: Worst case design bending moment. M_{Ed}^- : *0.04* kN·m

The design bending moment resistance $M_{c,Rd}$ is given by:

$$M_{c,Rd} = W_{pl,z} \cdot f_{yd}$$

$M_{c,Rd}$: *8.43* kN·m

Where:

Class: Section class, depending on its deformation capacity and development of plastic resistance of the flat elements of a section submitted to simple bending. **Class** : *2*

$W_{pl,z}$: Plastic strength modulus corresponding to the fibre with greatest tension, for class 1 and 2 sections. $W_{pl,z}$: *203.80* cm³

f_{yd}: Steel design strength. f_{yd} : *41.35* MPa

$$f_{yd} = f_{y,\theta} / \gamma_{M,\theta}$$

where:

$f_{y,\theta}$: Reduced elastic limit for the temperature reached by the section. $f_{y,\theta}$: *41.35* MPa

$$f_{y,\theta} = f_y \cdot k_{y,\theta}$$

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) f_y : *235.00* MPa

$k_{y,\theta}$: Elastic limit reduction factor for the temperature reached by the section. $k_{y,\theta}$: *0.18*

$\gamma_{M,\theta}$: Partial material safety factor. $\gamma_{M,\theta}$: *1.00*

Resistance to shear in the Z direction – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.6, and EN 1993-1-2: 2005, Article 4)

The following criteria must be satisfied:

$$\eta = \frac{V_{Ed}}{V_{c,Rd}} \le 1$$

η : *0.005* ✓

The worst case design forces occur for load combination SW

V_{Ed}: Worst case design shear force. V_{Ed} : *0.22* kN

The shear resistance $V_{c,Rd}$ is given by:

$$V_{c,Rd} = A_V \cdot \frac{f_{yd}}{\sqrt{3}}$$

$V_{c,Rd}$: *43.09* kN

Where:

A_V: Transverse shear area. A_V : *18.05* cm²

$$A_V = h \cdot t_w$$

Where:

h: Height of the section.	h :	*190.00*	mm
t_w: Web thickness.	t_w :	*6.50*	mm
f_{yd}: Steel design strength.	f_{yd} :	*41.35*	MPa

$$\mathbf{f_{yd}} = f_{y,\theta} / \gamma_{M,\theta}$$

Where:

$f_{y,\theta}$: Reduced elastic limit for the temperature reached by the section.	$f_{y,\theta}$:	*41.35*	MPa

$$\mathbf{f_{y,\theta}} = f_y \cdot k_{y,\theta}$$

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1)	f_y :	*235.00*	MPa
$k_{y,\theta}$: Elastic limit reduction factor for the temperature reached by the section.	$k_{y,\theta}$:	*0.18*	
$\gamma_{M,\theta}$: Partial material safety factor.	$\gamma_{M,\theta}$:	*1.00*	

Shear buckling of the web: (Eurocode 3 EN 1993-1-5: 2006, Article5)
Even though transverse stiffeners have not been provided, it is not necessary to check the buckling resistance of the web, as the following is verified:

$$\frac{\mathbf{d}}{\mathbf{t_w}} < \frac{72}{\eta} \cdot \varepsilon$$

20.62 < 60.00 ✓

Where:

λ_w: Slenderness of the web.	λ_w :	*20.62*

$$\lambda_{\mathbf{w}} = \frac{d}{t_w}$$

λ_{max}: Maximum slenderness.	λ_{max} :	*60.00*

$$\lambda_{\mathbf{max}} = \frac{72}{\eta} \cdot \varepsilon$$

η: Coefficient which allows to consider the additional resistance in plastic regime because of hardening due to deformed material.	η :	*1.20*
ε: Reduction factor.	ε :	*1.00*

$$\varepsilon = \sqrt{\frac{f_{ref}}{f_y}}$$

Where:

f_{ref}: Reference elastic limit.	f_{ref} :	*235.00*	MPa
f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1)	f_y :	*235.00*	MPa

<u>Resistance to shear in the Y direction – In case of fire</u> (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.6, and EN 1993-1-2: 2005, Article 4)

The following criteria must be satisfied:

$$\eta = \frac{V_{Ed}}{V_{c,Rd}} \le 1$$

$\eta <$ ***0.001*** ✓

The most unfavorable case of force design for load combination SW.

V_{Ed}: Worst case design shear force. **V_{Ed}** : *0.02* kN

The shear resistance $V_{c,Rd}$ is given by:

$$V_{c,Rd} = A_V \cdot \frac{f_{yd}}{\sqrt{3}}$$ $V_{c,Rd}$: 102.05 kN

Where:

A_v: Transverse shear area. A_v : 42.75 cm²

$$A_V = A - d \cdot t_w$$

Where:

A: Area of the gross section. **A** : 53.80 cm²

d: Height of the web. **d** : 170.00 mm

t_w: Web thickness t_w : 6.50 mm

f_{yd}: Steel design strength. f_{yd} : 41.35 MPa

$$f_{yd} = f_{y,\theta} / \gamma_{M,\theta}$$

Where:

$f_{y,\theta}$: Reduced elastic limit for the temperature reached by the section. $f_{y,\theta}$: 41.35 MPa

$$f_{y,\theta} = f_y \cdot k_{y,\theta}$$

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) f_y : 235.00 MPa

$k_{y,\theta}$: Elastic limit reduction factor for the temperature reached by the section. $k_{y,\theta}$: 0.18

$\gamma_{M,\theta}$: Partial material safety factor. $\gamma_{M,\theta}$: 1.00

Combined bending moment Y and shear force Z resistance – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.8, and EN 1993-1-2: 2005, Article 4)

It is not necessary to reduce the design bending resistance, as the worst case shear force V_{Ed} is not greater than 50% of the design shear resistance $V_{c,Rd}$.

$$V_{Ed} \le \frac{V_{c,Rd}}{2}$$ **0.22 kN ≤ 21.54 kN** ✓

The worst case design forces occur for load combination Sw

V_{Ed}: Worst case design shear force. V_{Ed} : 0.22 kN

$V_{c,Rd}$: Design resistant shear force. $V_{c,Rd}$: 43.09 kN

Combined bending moment Z and shear force Y resistance – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.8, and EN 1993-1-2: 2005, Article 4)

It is not necessary to reduce the design bending resistance, as the worst case shear force V_{Ed} is not greater than 50% of the design shear resistance $V_{c,Rd}$.

$$\mathbf{V_{Ed}} \le \frac{\mathbf{V_{c,Rd}}}{2}$$ **0.02 kN ≤ 51.03 kN** ✓

The worst case design force occurs at a point situated at node N1, for load combination SW

$\mathbf{V_{Ed}}$: Rasti me i disfavorshem ne projektimin e forces prerese.. $\mathbf{V_{Ed}}$: *0.02* kN

$\mathbf{V_{c,Rd}}$: Design resistant shear force. $\mathbf{V_{c,Rd}}$: *102.05* kN

Combined bending and axial resistance –In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.9, and EN 1993-1-2: 2005, Article 4)

The following criteria must be satisfied:

$$\eta = \left[\frac{M_{y,Ed}}{M_{N,Rd,y}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,Rd,z}}\right]^{\beta} \le 1$$ η : ***0.006*** ✓

$$\eta = \frac{N_{c,Ed}}{\chi_{min} \cdot A \cdot f_{yd}} + k_y \cdot \frac{M_{y,Ed}}{W_{pl,y} \cdot f_{yd}} + k_z \cdot \frac{M_{z,Ed}}{W_{pl,z} \cdot f_{yd}} \le 1$$ η : ***0.078*** ✓

$$\eta = \frac{N_{c,Ed}}{\chi_z \cdot A \cdot f_{yd}} + k_{LT} \cdot \frac{M_{y,Ed}}{\chi_{LT} \cdot W_{pl,y} \cdot f_{yd}} + k_z \cdot \frac{M_{z,Ed}}{W_{pl,z} \cdot f_{yd}} \le 1$$ η : ***0.078*** ✓

The worst case design force occurs at a point situated at node N3, for load combination SW

Where:

$\mathbf{N_{c,Ed}}$: Compressive axial force to be withstood from the analysis. $\mathbf{N_{c,Ed}}$: *7.47* kN

$\mathbf{M_{y,Ed}}$, $\mathbf{M_{z,Ed}}$: Worst case bending moments, in accordance with the Y and Z axes, respectively. $\mathbf{M_{y,Ed}}^{+}$: *0.40* kN·m

$\mathbf{M_{z,Ed}}^{-}$: *0.04* kN·m

Class: Section class, according to its deformation capacity and plastic resistance development of its flat elements, for axial load and simple bending. **Class** : *2*

$\mathbf{M_{N,Rd,y}}$, $\mathbf{M_{N,Rd,z}}$: Reduced design plastic resistant bending moments, about the Y and Z axes, respectively. $\mathbf{M_{N,Rd,y}}$: *17.76* kN·m

$\mathbf{M_{N,Rd,z}}$: *8.43* kN·m

$$\mathbf{M_{N,Rd,y}} = M_{pl,Rdy} \cdot (1-n)/(1-0.5 \cdot a) \le M_{pl,Rd,y}$$

$$n \le a \rightarrow \mathbf{M_{N,Rd,z}} = M_{pl,Rd,z}$$

$$\alpha = 2 \; ; \; \beta = 5 \cdot n \ge 1$$ α : *2.000*

β : *1.000*

Where:

$$\mathbf{n} = N_{c,Ed}/N_{pl,Rd}$$ $\mathbf{n}$: *0.034*

$\mathbf{N_{pl,Rd}}$: resistance of the gross section. $\mathbf{N_{pl,Rd}}$: *222.45* kN

$\mathbf{M_{pl,Rd,y}}$, $\mathbf{M_{pl,Rd,z}}$: Bending resistance of the gross section in plastic $\mathbf{M_{pl,Rd,y}}$: *17.76* kN·m conditions, with respect to the Y and Z axes, respectively. **$M_{pl,Rd,z}$** : *8.43* kN·m

$$\mathbf{a} = (A - 2 \cdot b \cdot t_f)/A \leq 0.5$$

a : *0.26*

A: Area of the gross section. **A** : *53.80* cm²

b: Flange width. **b** : *20.00* cm

t_f: Thickness of the flange. **t_f** : *10.00* mm

Buckling resistance: (Eurocode 3 EN 1993-1-1: 2005, Article 6.3.3)

A: Area of the gross section. **A** : *53.80* cm²

$W_{pl,y}$, $W_{pl,z}$: Plastic resistance modul corresponding to the fibre with greatest stress about the Y and Z axes, respectively. **$W_{pl,y}$** : *429.50* cm³ **$W_{pl,z}$** : *203.80* cm³

f_{yd}: Steel design strength **f_{yd}** : *41.35* MPa

$$\mathbf{f_{yd}} = f_{y,\theta}/\gamma_{M,\theta}$$

where:

$f_{y,\theta}$: Reduced elastic limit for the temperature reached by the section. **$f_{y,\theta}$** : *41.35* MPa

$$\mathbf{f_{y,\theta}} = f_y \cdot k_{y,\theta}$$

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) **f_y** : *235.00* MPa

$k_{y,\theta}$: Elastic limit reduction factor for the temperature reached by the section. **$k_{y,\theta}$** : *0.18*

$\gamma_{M,\theta}$: Partial material safety factor. **$\gamma_{M,\theta}$** : *1.00*

k_y, k_z, k_{LT}: Interaction coefficients.

$$\mathbf{k_y} = 1 - \frac{\mu_y \cdot N_{Ed}}{\chi_y \cdot A \cdot f_{yd}} \leq 3$$

k_y : *1.01*

$$\mathbf{k_z} = 1 - \frac{\mu_z \cdot N_{Ed}}{\chi_z \cdot A \cdot f_{yd}} \leq 3$$

k_z : *1.03*

$$\mathbf{k_{LT}} = 1 - \frac{\mu_{LT} \cdot N_{Ed}}{\chi_z \cdot A \cdot f_{yd}} \leq 1$$

k_{LT} : *1.00*

μ_y, μ_z, μ_{LT}: Auxiliary terms

$$\mu_y = (2 \cdot \beta_{M,y} - 5) \cdot \bar{\lambda}_y + 0.44 \cdot \beta_{M,y} + 0.29 \leq 0.8 \ ; \ \bar{\lambda}_y \leq 1.1$$

μ_y : *-0.35*

$$\mu_z = (1.2 \cdot \beta_{M,z} - 3) \cdot \bar{\lambda}_z + 0.71 \cdot \beta_{M,z} - 0.29 \leq 0.8$$

μ_z : *-0.65*

$$\mu_{LT} = 0{,}15 \cdot \bar{\lambda}_z \cdot \beta_{M,LT} - 0.15 \leq 0.9$$

μ_{LT} : *-0.06*

$\beta_{M,y}$, $\beta_{M,z}$, $\beta_{M,LT}$: Equivalent factors of uniform bending moment	$\beta_{M,y}$:	*1.00*
	$\beta_{M,z}$:	*1.00*
	$\beta_{M,LT}$:	*1.00*
χ_{min}: Minimum reduction coefficient due to the connection between χ_y and χ_z.	χ_{min} :	*0.66*
χ_y, χ_z: Connection reduction coefficients, for the Y and Z axes, respectively.	χ_y :	*0.79*
	χ_z :	*0.66*
χ_{LT}: Lateral reduction coefficient	χ_{LT} :	*1.00*
$\bar{\lambda}_y$, $\bar{\lambda}_z$: reduction of the space between the Y and Z axes, respectively.	$\bar{\lambda}_y$:	*0.36*
	$\bar{\lambda}_z$:	*0.60*

Combined bending, axial and shear resistance – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.10, and EN 1993-1-2: 2005, Article 4)

It is not necessary to reduce the design bending and axial force resistance, as the buckling effect can be ignored due to shear. Additionally, the worst case design shear force $\mathbf{V_{Ed}}$ is less than or equal to 50% of the design shear resistance $\mathbf{V_{c,Rd}}$.

The worst case design forces occur for load combination SW

$$\mathbf{V_{Ed,y}} \leq \frac{V_{c,Rd,y}}{2} \qquad 0.02\ \text{kN} \leq 51.03\ \text{kN}\ ✓$$

Where:

$V_{Ed,y}$: Rasti me i disfavorshem ne projektim per forcen prerese.	$V_{Ed,y}$:	*0.02*	kN
$V_{c,Rd,y}$: Design resistant shear force.	$V_{c,Rd,y}$:	*102.05*	kN

Torsional resistance – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.7, and EN 1993-1-2: 2005, Article 4)

The control does not continue, as there is no torsional moment

Combined Z shear and torsional resistance – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.7, and EN 1993-1-2: 2005, Article 4)

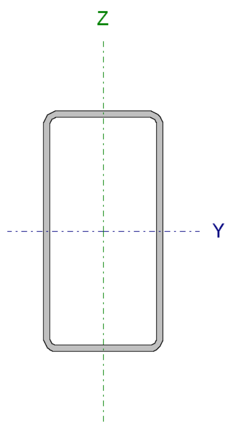
There is no interaction between the torsional moment and the shear force for any combination. Therefore the control is not done

Combined Y shear and torsional resistance – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.7, and EN 1993-1-2: 2005, Article 4)

There is no interaction between the torsional moment and the shear force for any combination. Therefore the control is not done

Bar N1/N2 (and for all nodes)

Seksioni: RHS 200 x100x6 te salduar (I vazhdueshem)
Materiali: Celik (Fe360)



Nyjet		Gjatesia (m)	Karakteristikat mekanike			
Fillestare	Finale		Sip. (cm²)	I_y(1) (cm4)	I_z(1) (cm4)	I_t(2) (cm4)
N1	N2	2.000	33.61	1699.23	574.94	1415.92

Shenime:
(1) Inercia në lidhje me aksin e treguar
(2) Momenti uniform torsional i inercisë

	Lidhja		Lidhja anesore	
	Plani XY	Plani XZ	Siper	Poshte
β	1.00	1.00	0.00	0.00
L_K	2.000	2.000	0.000	0.000
C_m	1.000	1.000	1.000	1.000
C_1	-		1.000	

Notation:
β: Koeficienti i lidhjes
L_K: Gjatesia e lidhjes (m)
C_m: Koeficienti i momentit
C_1: Faktori i modifikimit ne momentin kritik

Fire situation

Rezistenca e kerkuar: R 30
Faktori i formes: 153.48 m-1
Temperatura maksimale e shufres: 817.0 °C

Crushing of the web induced by the compressed flange – Outside temperature (Eurocode 3 EN 1993-1-5: 2006, Article 8)

The following criteria must be satisfied:

$$\frac{h_w}{t_w} \le k \frac{E}{f_{yf}} \sqrt{\frac{A_w}{A_{fc,ef}}}$$

$26.15 \le 199.27$ ✓

Where:

h_w: Height of the web.	h_w	:	*170.00*	mm
A_w: Area of the web.	t_w	:	*6.50*	mm
$A_{fc,ef}$: Reduced area of the compressed flange.	A_w	:	*11.05*	cm²
k: Coefficient which depends on the class of the section.	$A_{fc,ef}$	:	*20.00*	cm²
E: Modulus of Elasticity.	k	:	*0.30*	
f_{yf}: Steel elastic limit of the compressed flange.	E	:	*210000*	MPa
Where:	f_{yf}	:	*235.00*	MPa

$f_{yf} = f_y$

t_w: Web thickness.

Resistance to axial tension - Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.3)

The check does not proceed, as there is no tensile axial force.

Compression resistance - Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.4)

The following criteria must be satisfied:

$$\eta = \frac{N_{c,Ed}}{N_{c,Rd}} \le 1$$

η < *0.001* ✓

The worst case design force occurs at a point situated at node N1, for load combination 1.35*SW

$N_{c,Ed}$: Worst case design compressive axial force. $N_{c,Ed}$: *0.13* kN

The normal design compression force $\mathbf{N_{c,Rd}}$ should be taken as

$$\mathbf{N_{c,Rd}} = A \cdot f_{yd}$$

$N_{c,Rd}$: *1264.30* kN

Where:

Class: Section class, depending on its deformation capacity and development of plastic resistance of the compressed elements of a section. Class : *1*

A: Area of the gross section for class 1, 2 and 3 sections. A : *53.80* cm²

$\mathbf{f_{yd}}$: Forca e projektimit të çelikut. f_{yd} : *235.00* MPa

$$\mathbf{f_{yd}} = f_y / \gamma_{M0}$$

Where:

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) f_y : *235.00* MPa

γ_{M0}: Partial safety factor of the material γ_{M0} : *1.00*

Buckling resistance: (Eurocode 3 EN 1993-1-1: 2005, Article 6.3.1)

If the slenderness $\bar{\lambda} \le 0.2$ or the ratio $\mathbf{N_{c,Ed} / N_{cr}} \le 0.04$, the buckling effect can be ignored and only the resistance of the transverse section has to be checked.

$\bar{\lambda}$: Reduced slenderness. $\bar{\lambda}$: *0.31*

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}}$$

$N_{c,Ed}/N_{cr}$: Axial force ratio. $N_{c,Ed}/N_{cr}$: *0.000*

Where:

A:Siperfaqja e seksioneve bruto per seksionet 1, 2 dhe 3. A : *53.80* cm²

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) f_y : *235.00* MPa

N_{cr}: : Critical elastic buckling axial force, obtained from the smallest of the following values: N_{cr} : *13066.22* kN

Y – Axis bending resistance -Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.5)

The following criteria must be satisfied:

$$\eta = \frac{M_{Ed}}{M_{c,Rd}} \le 1$$

η : ***0.004*** ✓

For positive bending:

The worst case design force occurs at a point situated at a distance of 1.210 m from node N3, for load combination 1.35·SW.

$\mathbf{M_{Ed}}^+$: Worst case design bending moment. $\mathbf{M_{Ed}}^+$: *0.45* kN·m

For negative bending:

$\mathbf{M_{Ed}}^-$: Worst case design bending moment. $\mathbf{M_{Ed}}^-$: *0.00* kN·m

The design bending moment resistance $\mathbf{M_{c,Rd}}$ is given by:

$$\mathbf{M_{c,Rd}} = W_{pl,y} \cdot f_{yd}$$ $\mathbf{M_{c,Rd}}$: *100.93* kN·m

where:

Class: Section class, depending on its deformation capacity and development of plastic resistance of the flat elements of a section submitted to simple bending. **Class** : *1*

$\mathbf{W_{pl,y}}$: Plastic strength modulus corresponding to the fibre with greatest tension, for class 1 and 2 sections. $\mathbf{W_{pl,y}}$: *429.50* cm³

$\mathbf{f_{yd}}$: Steel design strength. $\mathbf{f_{yd}}$: *235.00* MPa

$$\mathbf{f_{yd}} = f_y / \gamma_{M0}$$

where:

$\mathbf{f_y}$: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) $\mathbf{f_y}$: *235.00* MPa

γ_{M0}: Partial safety factor of the material. γ_{M0} : *1.00*

Buckling resistance: (Eurocode 3 EN 1993-1-1: 2005, Article 6.3.2)

It does not continue, since the side link lengths are invalid.

Y - Axis bending resistance – Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.5)

The following criteria must be satisfied:

$$\eta = \frac{M_{Ed}}{M_{c,Rd}} \leq 1$$ $\eta <$ ***0.001*** ✓

For positive bending:

The worst case design force occurs at a point situated at a distance of 1.210 m from node N3, for load combination 1.35·SW.

$\mathbf{M_{Ed}}^+$: Worst case design bending moment. $\mathbf{M_{Ed}}^+$: *0.00* kN·m

For negative bending:

$\mathbf{M_{Ed}}^-$: Worst case design bending moment. $\mathbf{M_{Ed}}^-$: *0.00* kN·m

The design bending moment resistance $\mathbf{M_{c,Rd}}$ is given by:

$$\mathbf{M_{c,Rd}} = W_{pl,z} \cdot f_{yd}$$ $\mathbf{M_{c,Rd}}$: *47.89* kN·m

Where:

Class: Section class, depending on its deformation capacity and development of plastic resistance of the flat elements of a section submitted to simple bending. **Class** : *1*

$W_{pl,y}$: Plastic strength modulus corresponding to the fibre with greatest tension, for class 1 and 2 sections — **$W_{pl,z}$** : *203.80* cm³

f_{yd}: Steel design strength. — **f_{yd}** : *235.00* MPa

$$\mathbf{f_{yd}} = f_y / \gamma_{M0}$$

Where:

f_y: Reistenca e rrjedhshmerise. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) — **f_y** : *235.00* MPa

γ_{M0}: Yield strength — **γ_{M0}** : *1.00*

<u>Resistance to shear in the Z direction</u> <u>- Outside temperature</u> (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.6)

The following criteria must be satisfied:

$$\eta = \frac{V_{Ed}}{V_{c,Rd}} \leq 1$$

η : *0.003* ✓

The worst case design force occurs at node N3, for load combination 1.35·SW.

V_{Ed}: Worst case design shear force. — V_{Ed} : *0.68* kN

The shear resistance **$V_{c,Rd}$** is given by:

$$\mathbf{V_{c,Rd}} = A_V \cdot \frac{f_{yd}}{\sqrt{3}}$$

$V_{c,Rd}$: *244.90* kN

Where:

A_v: Transverse shear area. — A_v : *18.05* cm²

$$\mathbf{A_V} = h \cdot t_w$$

where:

h: Height of the web. — h : *190.00* mm

t_w: Web thickness. — t_w : *6.50* mm

f_{yd}: Steel design strength. — f_{yd} : *235.00* MPa

$$\mathbf{f_{yd}} = f_y / \gamma_{M0}$$

where:

f_y: Yield strength (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) — f_y : *235.00* MPa

γ_{M0}: Partial safety factor of the material. — γ_{M0} : *1.00*

Shear buckling of the web: (Eurocode 3 EN 1993-1-5: 2006, Article5)
Even though transverse stiffeners have not been provided, it is not necessary to check the buckling resistance of the web, as the following is verified:

$$\frac{\mathbf{d}}{\mathbf{t_w}} < \frac{72}{\eta} \cdot \varepsilon$$

20.62 < 60.00 ✓

λ_w: Slenderness of the web. — λ_w : *20.62*

$$\lambda_w = \frac{d}{t_w}$$

λ_{max}: Maximum slenderness. — λ_{max} : *60.00*

$$\lambda_{max} = \frac{72}{\eta} \cdot \varepsilon$$

η: Coefficient which allows to consider the additional resistance in plastic regime because of hardening due to deformed material. η : *1.20*

ε: Reduction factor. ε : *1.00*

$$\varepsilon = \sqrt{\frac{f_{ref}}{f_y}}$$

where:

f_{ref}: Reference elastic limit. f_{ref} : *235.00* MPa

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) f_y : *235.00* MPa

Resistance to shear in the Y direction Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.6)

The following criteria must be satisfied:

$$\eta = \frac{V_{Ed}}{V_{c,Rd}} \leq 1$$

$\eta <$ ***0.001*** ✓

The most unfavorable case of force design for load combination 1.35 · SW.

$\mathbf{V_{Ed}}$: Worst case design shear force. $\mathbf{V_{Ed}}$: *0.00* kN

The shear resistance $\mathbf{V_{c,Rd}}$ is given by:

$$\mathbf{V_{c,Rd}} = A_V \cdot \frac{f_{yd}}{\sqrt{3}}$$

$\mathbf{V_{c,Rd}}$: *580.02* kN

where:

$\mathbf{A_v}$: Transverse shear area. $\mathbf{A_v}$: *42.75* cm²

$$\mathbf{A_V} = A - d \cdot t_w$$

where:

A: Area of the gross section. **A** : *53.80* cm²

d: Height of the web. **d** : *170.00* mm

$\mathbf{t_w}$: Web thickness. $\mathbf{t_w}$: *6.50* mm

$\mathbf{f_{yd}}$: Steel design strength. $\mathbf{f_{yd}}$: *235.00* MPa

$$\mathbf{f_{yd}} = f_y / \gamma_{M0}$$

where:

$\mathbf{f_y}$: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) $\mathbf{f_y}$: *235.00* MPa

γ_{M0}: Partial safety factor of the material γ_{M0} : *1.00*

Combined bending moment Y and shear force Z resistance – Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.8)

It is not necessary to reduce the design bending resistance, as the worst case shear force $\mathbf{V_{Ed}}$ is not greater than 50% of the design shear resistance $\mathbf{V_{c,Rd}}$.

$$\mathbf{V_{Ed}} \le \frac{\mathbf{V_{c,Rd}}}{2}$$

0.68 kN ≤ 122.45 kN ✓

The worst case design forces occurs at node N3 for load combination 1.35SW.

V_{Ed}: Worst case design shear force. **V_{Ed}** : *0.68* kN

$V_{c,Rd}$: Design resistant shear force. **$V_{c,Rd}$** : *244.90* kN

Combined bending moment Y and shear force Z - Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.8)

It is not necessary to reduce the design bending resistance, as the worst case shear force **V_{Ed}** is not greater than 50% of the design shear resistance **$V_{c,Rd}$**.

$$\mathbf{V_{Ed}} \le \frac{\mathbf{V_{c,Rd}}}{2}$$

0.00 kN ≤ 290.01 kN ✓

The worst case design forces occurs at node N3 for load combination 1.35SW..

V_{Ed}: Worst case design shear force. **V_{Ed}** : *0.00* kN

$V_{c,Rd}$: Design resistant shear force. **$V_{c,Rd}$** : *580.02* kN

Combined bending and axial resistance - Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.9)

The following criteria must be satisfied:

$$\eta = \frac{M_{y,Ed}}{M_{N,Rd,y}} \le 1$$

η : ***0.004*** ✓

$$\eta = \frac{N_{c,Ed}}{\chi_y \cdot A \cdot f_{yd}} + k_{yy} \cdot \frac{M_{y,Ed}}{\chi_{LT} \cdot W_{pl,y} \cdot f_{yd}} + k_{yz} \cdot \frac{M_{z,Ed}}{W_{pl,z} \cdot f_{yd}} \le 1$$

η : ***0.005*** ✓

$$\eta = \frac{N_{c,Ed}}{\chi_z \cdot A \cdot f_{yd}} + k_{zy} \cdot \frac{M_{y,Ed}}{\chi_{LT} \cdot W_{pl,y} \cdot f_{yd}} + k_{zz} \cdot \frac{M_{z,Ed}}{W_{pl,z} \cdot f_{yd}} \le 1$$

η : ***0.002*** ✓

The worst case design force occurs at a point situated at a distance of 1.210 m from node N3, for load combination 1.35·SW.

Where:

$N_{c,Ed}$: Compressive axial force to be withstood from the analysis. **$N_{c,Ed}$** : *0.09* kN

$M_{y,Ed}$, $M_{z,Ed}$ Worst case bending moments, in accordance with the Y and Z axes, respectively. **$M_{y,Ed}^+$** : *0.45* kN·m

$M_{z,Ed}^+$: *0.00* kN·m

Class: Section class, according to its deformation capacity and plastic resistance development of its flat elements, for axial load and simple bending. **Class** : 1

$\mathbf{M_{N,Rd,y}}$: Reduced design plastic resistant bending moments, about the Y and Z axes, respectively. $\mathbf{M_{N,Rd,y}}$: 100.93 kN·m

$$\mathbf{M_{N,Rd,y}} = M_{pl,Rdy} \cdot (1-n)/(1-0.5\cdot a) \le M_{pl,Rd,y}$$

Where:

$$\mathbf{n} = N_{c,Ed}/N_{pl,Rd}$$

n : 0.000

$\mathbf{N_{pl,Rd}}$: resistance of the gross section. $\mathbf{N_{pl,Rd}}$: 1264.30 kN

$\mathbf{M_{pl,Rd,y}}$ Bending resistance of the gross section in plastic conditions, with respect to the Y axes $\mathbf{M_{pl,Rd,y}}$: 100.93 kN·m

$$\mathbf{a} = (A - 2\cdot b\cdot t_f)/A \le 0.5$$

a : 0.26

A: Area of the gross section. **A** : 53.80 cm²

b: Flange width. **b** : 20.00 cm

$\mathbf{t_f}$: Thickness of the flange. $\mathbf{t_f}$: 10.00 mm

Buckling resistance: (Eurocode 3 EN 1993-1-1: 2005, Article 6.3.3)

A: Area of the gross section. **A** : 53.80 cm²

$\mathbf{W_{pl,y}}$, $\mathbf{W_{pl,z}}$: Plastic resistance moduli corresponding to the fibre with greatest stress about the Y and Z axes, respectively. $\mathbf{W_{pl,y}}$: 429.50 cm³ $\mathbf{W_{pl,z}}$: 203.80 cm³

$\mathbf{f_{yd}}$: Steel design strength. $\mathbf{f_{yd}}$: 235.00 MPa

$$\mathbf{f_{yd}} = f_y/\gamma_{M1}$$

Where:

$\mathbf{f_y}$: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) $\mathbf{f_y}$: 235.00 MPa

$\boldsymbol{\gamma_{M1}}$: Partial safety factor of the material. $\boldsymbol{\gamma_{M1}}$: 1.00

$\mathbf{K_{yy}}$, $\mathbf{K_{yz}}$, $\mathbf{K_{zy}}$, $\mathbf{K_{zz}}$: Interaction coefficients.

$$\mathbf{k_{yy}} = C_{m,y}\cdot C_{m,LT}\cdot \frac{\mu_y}{1-\frac{N_{Ed}}{N_{cr,y}}}\cdot \frac{1}{C_{yy}}$$

$\mathbf{K_{yy}}$: 1.00

$$\mathbf{k_{yz}} = C_{m,z}\cdot \frac{\mu_y}{1-\frac{N_{Ed}}{N_{cr,z}}}\cdot \frac{1}{C_{yz}}\cdot 0.6\cdot\sqrt{\frac{w_z}{w_y}}$$

$\mathbf{K_{yz}}$: 0.70

$$\mathbf{k_{zy}} = C_{m,y}\cdot C_{m,LT}\cdot \frac{\mu_z}{1-\frac{N_{Ed}}{N_{cr,y}}}\cdot \frac{1}{C_{zy}}\cdot 0.6\cdot\sqrt{\frac{w_y}{w_z}}$$

$\mathbf{K_{zy}}$: 0.52

$$\mathbf{k_{zz}} = C_{m,z}\cdot \frac{\mu_z}{1-\frac{N_{Ed}}{N_{cr,z}}}\cdot \frac{1}{C_{zz}}$$

$\mathbf{K_{zz}}$: 1.00

Auxiliary terms:

$$\mu_y = \frac{1 - \frac{N_{Ed}}{N_{cr,y}}}{1 - \chi_y \cdot \frac{N_{Ed}}{N_{cr,y}}}$$ μ_y : 1.00

$$\mu_z = \frac{1 - \frac{N_{Ed}}{N_{cr,z}}}{1 - \chi_z \cdot \frac{N_{Ed}}{N_{cr,z}}}$$ μ_z : 1.00

$$C_{yy} = 1 + (w_y - 1) \cdot \left[\left(2 - \frac{1.6}{w_y} \cdot C_{my}^2 \cdot \bar{\lambda}_{max} - \frac{1.6}{w_y} \cdot C_{my}^2 \cdot \bar{\lambda}_{max}^2\right) \cdot n_{pl} - b_{LT}\right] \geq \frac{W_{el,y}}{W_{pl,y}}$$ C_{yy} : 1.00

$$C_{yz} = 1 + (w_z - 1) \cdot \left[\left(2 - 14 \cdot \frac{C_{mz}^2 \cdot \bar{\lambda}_{max}^2}{w_z^5}\right) \cdot n_{pl} - c_{LT}\right] \geq 0.6 \cdot \sqrt{\frac{w_z}{w_y}} \cdot \frac{W_{el,z}}{W_{pl,z}}$$ C_{yz} : 1.00

$$C_{zy} = 1 + (w_y - 1) \cdot \left[\left(2 - 14 \cdot \frac{C_{my}^2 \cdot \bar{\lambda}_{max}^2}{w_y^5}\right) \cdot n_{pl} - d_{LT}\right] \geq 0.6 \cdot \sqrt{\frac{w_y}{w_z}} \cdot \frac{W_{el,y}}{W_{pl,y}}$$ C_{zy} : 1.00

$$C_{zz} = 1 + (w_z - 1) \cdot \left[\left(2 - \frac{1.6}{w_z} \cdot C_{mz}^2 \cdot \bar{\lambda}_{max} - \frac{1.6}{w_z} \cdot C_{mz}^2 \cdot \bar{\lambda}_{max}^2 - e_{LT}\right) \cdot n_{pl}\right] \geq \frac{W_{el,z}}{W_{pl,z}}$$ C_{zz} : 1.00

$$a_{LT} = 1 - \frac{I_t}{I_y} \geq 0$$ a_{LT} : 0.99

$$b_{LT} = 0.5 \cdot a_{LT} \cdot \bar{\lambda}_0^2 \cdot \frac{M_{y,Ed}}{\chi_{LT} \cdot M_{pl,Rd,y}} \cdot \frac{M_{z,Ed}}{M_{pl,Rd,z}}$$ b_{LT} : 0.00

$$c_{LT} = 10 \cdot a_{LT} \cdot \frac{\bar{\lambda}_0^2}{5 + \bar{\lambda}_z^4} \cdot \frac{M_{y,Ed}}{C_{m,y} \cdot \chi_{LT} \cdot M_{pl,Rd,y}}$$ c_{LT} : 0.00

$$d_{LT} = 2 \cdot a_{LT} \cdot \frac{\bar{\lambda}_0}{0.1 + \bar{\lambda}_z^4} \cdot \frac{M_{y,Ed}}{C_{m,y} \cdot \chi_{LT} \cdot M_{pl,Rd,y}} \cdot \frac{M_{z,Ed}}{C_{m,z} \cdot M_{pl,Rd,z}}$$ d_{LT} : 0.00

$$e_{LT} = 1.7 \cdot a_{LT} \cdot \frac{\bar{\lambda}_0}{0.1 + \bar{\lambda}_z^4} \cdot \frac{M_{y,Ed}}{C_{m,y} \cdot \chi_{LT} \cdot M_{pl,Rd,y}}$$ e_{LT} : 0.00

$$w_y = \frac{W_{pl,y}}{W_{el,y}} \leq 1.5$$ w_y : 1.11

$$w_z = \frac{W_{pl,z}}{W_{el,z}} \leq 1.5$$ w_z : 1.50

$$n_{pl} = \frac{N_{Ed}}{N_{pl,Rd}}$$ n_{pl} : 0.00

Given that:

$$\bar{\lambda}_0 \leq 0.2 \cdot \sqrt{C_1} \cdot \sqrt[4]{\left(1-\frac{N_{Ed}}{N_{cr,y}}\right)\cdot\left(1-\frac{N_{Ed}}{N_{cr,T}}\right)}$$ **0.00** ≤ **0.20**

$$C_{m,y} = C_{m,y,0}$$ $C_{m,y}$: *1.00*

$$C_{m,z} = C_{m,z,0}$$ $C_{m,z}$: *1.00*

$$C_{m,LT} = 1.00$$ $C_{m,LT}$: *1.00*

$C_{m,y,0}$, $C_{m,z,0}$: Equivalent uniform bending moment factors.	$C_{m,y,0}$:	*1.00*
	$C_{m,z,0}$:	*1.00*
C_1: Factor which depends on the support conditions and bending moment envelope of the bar.	C_1 :	*1.00*
χ_y, χ_z: Koeficientet e reduktimit te lidhjes per akset Y dhe Z respektivisht	χ_y :	*1.00*
	χ_z :	*1.00*
χ_{LT}: Lateral buckling reduction coefficient.	χ_{LT} :	*1.00*
$\bar{\lambda}_{max}$: Maximum slenderness between $\bar{\lambda}_y$ and $\bar{\lambda}_z$.	$\bar{\lambda}_{max}$:	*0.31*
$\bar{\lambda}_y$, $\bar{\lambda}_z$: Reduced slendernesses with respect to the Y and Z axes, respectively.	$\bar{\lambda}_y$:	*0.31*
	$\bar{\lambda}_z$:	*0.23*
$\bar{\lambda}_{LT}$: Reduced slenderness.	$\bar{\lambda}_{LT}$:	*0.00*
$\bar{\lambda}_0$: Reduced slenderness, with respect to lateral buckling, for a uniform bending moment.	$\bar{\lambda}_0$:	*0.00*
$W_{el,y}$, $W_{el,z}$: Elastic resistant modules corresponding to the compressed fibre, about the Y and Z axes, respectively.	$W_{el,y}$:	*388.63* cm³
	$W_{el,z}$:	*133.60* cm³
N_{cr}: Critical elastic buckling axial force	N_{cr} :	*13066.22* kN
I_y: Moment of inertia of the gross section, with respect to the Y-axis.	I_y :	*7384.00* cm4
I_t: Uniform torsional moment of inertia.	I_t :	*41.96* cm4

Combined bending, axial and shear resistance – Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.10)

It is not necessary to reduce the design bending and axial force resistance, as the buckling effect can be ignored due to shear. Additionally, the worst case design shear force V_{Ed} is less than or equal to 50% of the design shear resistance $V_{c,Rd}$.

The worst case design forces occurs for load combination 1.35SW..

$$V_{Ed,y} \leq \frac{V_{c,Rd,y}}{2}$$ 0.00 kN ≤ 290.01 kN ✓

where:

$V_{Ed,y}$: Worst case design shear force. $V_{Ed,y}$: *0.00* kN

$V_{c,Rd,y}$: Design resistant shear force. $V_{c,Rd,y}$: *580.02* kN

Torsional resistance – Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.7)

The control does not continue, as there is no torsional moment

Combined Z shear and torsional resistance – Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.7)

There is no interaction between the torsional moment and the shear force for any combination. Therefore the control is not done

Combined Y shear and torsional resistance – Outside temperature (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.7)

There is no interaction between the torsional moment and the shear force for any combination. Therefore the control is not done

Axial tensile strength – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.3, and EN 1993-1-2: 2005, Article 4)

Control is not done, as there is no axial tensile force.

Compressive strength – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.4, and EN 1993-1-2: 2005, Article 4)

The following criteria must be satisfied:

$$\eta = \frac{N_{c,Ed}}{N_{c,Rd}} \leq 1$$

η < *0.001* ✓

$$\eta = \frac{N_{c,Ed}}{N_{b,Rd}} \leq 1$$

η < *0.001* ✓

The worst case design force occurs at node N3, for load combination SW

$N_{c,Ed}$: Worst case design compressive axial force. $N_{c,Ed}$: *0.09* kN

The normal design compression force $\mathbf{N_{c,Rd}}$ should be taken as:

$$\mathbf{N_{c,Rd}} = A \cdot f_{yd}$$

$N_{c,Rd}$: *254.18* kN

Where:

Class: Section class, depending on its deformation capacity and development of plastic resistance of the compressed elements of a Class : *2* section.

A: Area of the gross section for class 1, 2 and 3 sections.	A	:	*53.80*	cm²
f_{yd}: Steel design strength.	f_{yd}	:	*47.25*	MPa

$$\boxed{\mathbf{f_{yd}} = f_{y,\theta}/\gamma_{M,\theta}}$$

where:

$f_{y,\theta}$: Reduced elastic limit for the temperature reached by the section.	$f_{y,\theta}$	:	*47.25*	MPa

$$\mathbf{f_{y,\theta}} = f_y \cdot k_{y,\theta}$$

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1)	f_y	:	*235.00*	MPa
$k_{y,\theta}$: Elastic limit reduction factor for the temperature reached by the section.	$k_{y,\theta}$	:	*0.20*	
$\gamma_{M,\theta}$: Partial material safety factor.	$\gamma_{M,\theta}$	:	*1.00*	

Buckling resistance: (Eurocode 3 EN 1993-1-1: 2005, Article 6.3.1)
The calculated connection resistance Nb, Rd of a compression rod is given by:

$$\mathbf{N_{b,Rd}} = \chi \cdot A \cdot f_{yd}$$

	$N_{b,Rd}$	:	*205.04*	kN

where:

A: Area of the gross section for class 1, 2 and 3 sections	A	:	*53.80*	cm²
f_{yd}: Steel design strength.	f_{yd}	:	*47.25*	MPa

$$\mathbf{f_{yd}} = f_{y,\theta}/\gamma_{M,\theta}$$

where:

$f_{y,\theta}$: Reduced elastic limit for the temperature reached by the section.	$f_{y,\theta}$	:	*47.25*	MPa

$$\mathbf{f_{y,\theta}} = f_y \cdot k_{y,\theta}$$

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1)	f_y	:	*235.00*	MPa
$k_{y,\theta}$: Elastic limit reduction factor for the temperature reached by the section.	$k_{y,\theta}$	:	*0.20*	
$\gamma_{M,\theta}$: Partial material safety factor	$\gamma_{M,\theta}$	:	*1.00*	

χ: Buckling reduction coefficients.

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \left(\bar{\lambda}\right)^2}} \le 1$$

	χ_{FT}	:	*0.81*	

where:

$$\Phi = 0.5 \cdot \left[1 + \alpha \cdot \bar{\lambda} + \bar{\lambda}^2\right]$$

	ϕ_{FT}	:	*0.67*	
α_{FT}: Elastic imperfection coefficient.	α_{FT}	:	*0.65*	

$\bar{\lambda}$: Reduced slenderness

$$\bar{\lambda} = \sqrt{\frac{A \cdot f_y}{N_{cr}}}$$

	$\bar{\lambda}_{FT}$	:	*0.34*	
N_{cr}: Critical elastic buckling axial force	N_{cr}	:	*13066.22*	kN

Y – Axis bending resistance – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.5, and EN 1993-1-2: 2005, Article 4)

The following criteria must be satisfied:

$$\eta = \frac{M_{Ed}}{M_{c,Rd}} \leq 1$$

η : ***0.016*** ✓

For positive bending:

The worst case design force occurs at a point situated at a distance of 1.210 m from node N3, for load combination SW.

M_{Ed}^{+}: Worst case design bending moment — **M_{Ed}^{+}** : *0.33* kN·m

For negative bending:

M_{Ed}^{-}: Worst case design bending moment. — **M_{Ed}^{-}** : *0.00* kN·m

The design bending moment resistance **$M_{c,Rd}$** is given by:

$$\mathbf{M_{c,Rd}} = W_{pl,y} \cdot f_{yd}$$

$M_{c,Rd}$: *20.29* kN·m

where:

Class: Section class, depending on its deformation capacity and development of plastic resistance of the flat elements of a section submitted to simple bending. — **Class** : *2*

$W_{pl,y}$: Plastic strength modulus corresponding to the fibre with greatest tension, for class 1 and 2 sections. — **$W_{pl,y}$** : *429.50* cm³

f_{yd}: Steel design strength. — **f_{yd}** : *47.25* MPa

$$\mathbf{f_{yd}} = f_{y,\theta} / \gamma_{M,\theta}$$

where:

$f_{y,\theta}$: Reduced elastic limit for the temperature reached by the section. — **$f_{y,\theta}$** : *47.25* MPa

$$\mathbf{f_{y,\theta}} = f_y \cdot k_{y,\theta}$$

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) — **f_y** : *235.00* MPa

$k_{y,\theta}$: Elastic limit reduction factor for the temperature reached by the section. — **$k_{y,\theta}$** : *0.20*

$\gamma_{M,\theta}$: Partial material safety factor — **$\gamma_{M,\theta}$** : *1.00*

Lateral connection resistance: (Eurocode 3 EN 1993-1-1: 2005, Article 6.3.2)

It does not continue, since the side link lengths are invalid.

Z – Axis bending resistance – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.5, and EN 1993-1-2: 2005, Article 4)

The following criteria must be satisfied:

$$\eta = \frac{M_{Ed}}{M_{c,Rd}} \leq 1$$

η < ***0.001*** ✓

For positive bending:

The worst case design force occurs at node N3, for load combination SW.

M_{Ed}^{+}: Worst case design bending moment. **M_{Ed}^{+}** : *0.00* kN·m

For negative bending:

M_{Ed}^{-}: Worst case design bending moment. **M_{Ed}^{-}** : *0.00* kN·m

The design bending moment resistance **$M_{c,Rd}$** is given by:

$$\mathbf{M_{c,Rd}} = W_{pl,z} \cdot f_{yd}$$ **$M_{c,Rd}$** : *9.63* kN·m

where:

Class: Section class, depending on its deformation capacity and development of plastic resistance of the flat elements of a section submitted to simple bending. **Class** : *2*

$W_{pl,z}$: Plastic strength modulus corresponding to the fibre with greatest tension, for class 1 and 2 sections. **$W_{pl,z}$** : *203.80* cm³

f_{yd}: Steel design strength. **f_{yd}** : *47.25* MPa

$$\mathbf{f_{yd}} = f_{y,\theta} / \gamma_{M,\theta}$$

where:

$f_{y,\theta}$: Reduced elastic limit for the temperature reached by the section. **$f_{y,\theta}$** : *47.25* MPa

$$\mathbf{f_{y,\theta}} = f_y \cdot k_{y,\theta}$$

f_y: Yield strength (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) **f_y** : *235.00* MPa

$k_{y,\theta}$: Elastic limit reduction factor for the temperature reached by the section. **$k_{y,\theta}$** : *0.20*

$\gamma_{M,\theta}$: Partial material safety factor. **$\gamma_{M,\theta}$** : *1.00*

<u>Resistance to shear in the Z direction</u> <u>– In case of fire</u> (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.6, and EN 1993-1-2: 2005, Article 4)

The following criteria must be satisfied:

$$\eta = \frac{V_{Ed}}{V_{c,Rd}} \leq 1$$ η : *0.010* ✓

The worst case design forces occur at node N3 for load combination SW

V_{Ed}: Worst case design shear force. V_{Ed} : *0.50* kN

The shear resistance **$V_{c,Rd}$** is given by:

$$\mathbf{V_{c,Rd}} = A_V \cdot \frac{f_{yd}}{\sqrt{3}}$$ $V_{c,Rd}$: *49.24* kN

where:

A_v: Transverse shear area. A_v : 18.05 cm²

$$\mathbf{A_v} = h \cdot t_w$$

where:

h: Height of the section. h : 190.00 mm

t_w: Web thickness. t_w : 6.50 mm

f_{yd}: Steel design strength. f_{yd} : 47.25 MPa

$$\mathbf{f_{yd}} = f_{y,\theta} / \gamma_{M,\theta}$$

where:

$f_{y,\theta}$: Reduced elastic limit for the temperature reached by the section. $f_{y,\theta}$: 47.25 MPa

$$\mathbf{f_{y,\theta}} = f_y \cdot k_{y,\theta}$$

f_y: Yield strength (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) f_y : 235.00 MPa

$k_{y,\theta}$: Elastic limit reduction factor for the temperature reached by the section. $k_{y,\theta}$: 0.20

$\gamma_{M,\theta}$: Partial material safety factor. $\gamma_{M,\theta}$: 1.00

Shear buckling of the web :(Eurocode 3 EN 1993-1-5: 2006, Article5)
Even though transverse stiffeners have not been provided, it is not necessary to check the buckling resistance of the web, as the following is verified:

$$\frac{\mathbf{d}}{\mathbf{t_w}} < \frac{72}{\eta} \cdot \varepsilon$$

20.62 < 60.00 ✓

where:

λ_w: Hollesia e membranes. λ_w : 20.62

$$\lambda_w = \frac{d}{t_w}$$

λ_{max}: Slenderness of the web. λ_{max} : 60.00

$$\lambda_{max} = \frac{72}{\eta} \cdot \varepsilon$$

η: Coefficient which allows to consider the additional resistance in plastic regime because of hardening due to deformed material. η : 1.20

ε: Reduction factor. ε : 1.00

$$\varepsilon = \sqrt{\frac{f_{ref}}{f_y}}$$

where:

f_{ref}: Reference elastic limit. f_{ref} : 235.00 MPa

f_y: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) f_y : 235.00 MPa

Resistance to shear in the Y direction – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.6, and EN 1993-1-2: 2005, Article 4)

The following criteria must be satisfied:

$$\eta = \frac{V_{Ed}}{V_{c,Rd}} \leq 1$$

η < ***0.001*** ✓

The most unfavorable case of force design for load combination SW.

$\mathbf{V_{Ed}}$: Worst case design shear force. $\mathbf{V_{Ed}}$: 0.00 kN

The shear resistance $\mathbf{V_{c,Rd}}$ is given by:

$$\mathbf{V_{c,Rd}} = A_V \cdot \frac{f_{yd}}{\sqrt{3}}$$ $\mathbf{V_{c,Rd}}$: 116.61 kN

where:

$\mathbf{A_v}$: Transverse shear area. $\mathbf{A_v}$: 42.75 cm²

$$\mathbf{A_V} = A - d \cdot t_w$$

where:

$\mathbf{A}$: Area of the gross section. $\mathbf{A}$: 53.80 cm²

$\mathbf{d}$: Height of the web. $\mathbf{d}$: 170.00 mm

$\mathbf{t_w}$: Web thickness $\mathbf{t_w}$: 6.50 mm

$\mathbf{f_{yd}}$: Steel design strength. $\mathbf{f_{yd}}$: 47.25 MPa

$$\mathbf{f_{yd}} = f_{y,\theta}/\gamma_{M,\theta}$$

where:

$\mathbf{f_{y,\theta}}$: Reduced elastic limit for the temperature reached by the section. $\mathbf{f_{y,\theta}}$: 47.25 MPa

$$\mathbf{f_{y,\theta}} = f_y \cdot k_{y,\theta}$$

$\mathbf{f_y}$: Yield strength. (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) $\mathbf{f_y}$: 235.00 MPa

$\mathbf{k_{y,\theta}}$: Elastic limit reduction factor for the temperature reached by the section. $\mathbf{k_{y,\theta}}$: 0.20

$\gamma_{M,\theta}$: Partial material safety factor. $\gamma_{M,\theta}$: 1.00

Combined bending moment Y and shear force Z resistance – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.8, and EN 1993-1-2: 2005, Article 4)

It is not necessary to reduce the design bending resistance, as the worst case shear force $\mathbf{V_{Ed}}$ is not greater than 50% of the design shear resistance $\mathbf{V_{c,Rd}}$.

$$\mathbf{V_{Ed}} \le \frac{\mathbf{V_{c,Rd}}}{2}$$ **0.50 kN ≤ 24.62 kN** ✓

The worst case design forces occur at node N3 for load combination Sw

V_{Ed}: Worst case design shear force. **V_{Ed}** : *0.50* kN

$V_{c,Rd}$: Design resistant shear force. **$V_{c,Rd}$** : *49.24* kN

Combined bending moment Z and shear force Y resistance – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.8, and EN 1993-1-2: 2005, Article 4)

It is not necessary to reduce the design bending resistance, as the worst case shear force **V_{Ed}** is not greater than 50% of the design shear resistance **$V_{c,Rd}$**.

$$V_{Ed} \leq \frac{V_{c,Rd}}{2}$$

0.00 kN ≤ 58.31 kN ✓

The worst case design force occurs at a point situated at node N1, for load combination SW

V_{Ed}: Rasti me i disfavorshem ne projektimin e forces prerese **V_{Ed}** : *0.00* kN

$V_{c,Rd}$: Design resistant shear force. **$V_{c,Rd}$** : *116.61* kN

Combined bending and axial resistance –In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.9, and EN 1993-1-2: 2005, Article 4)

The following criteria must be satisfied:

$$\eta = \frac{M_{y,Ed}}{M_{N,Rd,y}} \leq 1$$

η : ***0.016*** ✓

$$\eta = \frac{N_{c,Ed}}{\chi_{min} \cdot A \cdot f_{yd}} + k_y \cdot \frac{M_{y,Ed}}{W_{pl,y} \cdot f_{yd}} + k_z \cdot \frac{M_{z,Ed}}{W_{pl,z} \cdot f_{yd}} \leq 1$$

η : ***0.017*** ✓

$$\eta = \frac{N_{c,Ed}}{\chi_z \cdot A \cdot f_{yd}} + k_{LT} \cdot \frac{M_{y,Ed}}{\chi_{LT} \cdot W_{pl,y} \cdot f_{yd}} + k_z \cdot \frac{M_{z,Ed}}{W_{pl,z} \cdot f_{yd}} \leq 1$$

η : ***0.017*** ✓

The worst case design force occurs at a point situated at a distance of 1.210 m from node N3, for load combination SW.

where:

$N_{c,Ed}$: Compressive axial force to be withstood from the analysis. **$N_{c,Ed}$** : *0.06* kN

$M_{y,Ed}$, $M_{z,Ed}$: Worst case bending moments, in accordance with the Y and **$M_{y,Ed}^{+}$** : *0.33* kN·m

Z axes, respectively. | $\mathbf{M_{z,Ed}}^{+}$: 0.00 kN·m

Klasa: Section class, according to its deformation capacity and plastic resistance development of its flat elements, for axial load and simple bending. | **Class** : 2

$\mathbf{M_{N,Rd,y}}$: Reduced design plastic resistant bending moments, about the Y and Z axes, respectively. | $\mathbf{M_{N,Rd,y}}$: 20.29 kN·m

$$\mathbf{M_{N,Rd,y}} = M_{pl,Rdy} \cdot (1-n)/(1-0.5 \cdot a) \le M_{pl,Rd,y}$$

Where:

$$\mathbf{n} = N_{c,Ed}/N_{pl,Rd}$$

n : 0.000

$\mathbf{N_{pl,Rd}}$: resistance of the gross section. | $\mathbf{N_{pl,Rd}}$: 254.18 kN

$\mathbf{M_{pl,Rd,y}}$: Bending resistance of the gross section in plastic conditions, with respect to the Y and Z axes, respectively. | $\mathbf{M_{pl,Rd,y}}$: 20.29 kN·m

$$\mathbf{a} = (A - 2 \cdot b \cdot t_f)/A \le 0.5$$

a : 0.26

A: Area of the gross section. | **A** : 53.80 cm²

b: Flange width. | **b** : 20.00 cm

$\mathbf{t_f}$: Thickness of the flange. | $\mathbf{t_f}$: 10.00 mm

Buckling resistance: (Eurocode 3 EN 1993-1-1: 2005, Article 6.3.3)

A: Area of the gross section. | **A** : 53.80 cm²

$\mathbf{W_{pl,y}}$, $\mathbf{W_{pl,z}}$ Plastic resistance modul corresponding to the fibre with greatest stress about the Y and Z axes, respectively. | $\mathbf{W_{pl,y}}$: 429.50 cm³ | $\mathbf{W_{pl,z}}$: 203.80 cm³

$\mathbf{f_{yd}}$: Steel design strength | $\mathbf{f_{yd}}$: 47.25 MPa

$$\mathbf{f_{yd}} = f_{y,\theta}/\gamma_{M,\theta}$$

where:

$\mathbf{f_{y,\theta}}$: Reduced elastic limit for the temperature reached by the section. | $\mathbf{f_{y,\theta}}$: 47.25 MPa

$$\mathbf{f_{y,\theta}} = f_y \cdot k_{y,\theta}$$

$\mathbf{f_y}$: Yield strength (Eurocode 3 EN 1993-1-1: 2005, Table 3.1) | $\mathbf{f_y}$: 235.00 MPa

$\mathbf{k_{y,\theta}}$: Elastic limit reduction factor for the temperature reached by the section. | $\mathbf{k_{y,\theta}}$: 0.20

$\gamma_{M,\theta}$: Partial material safety factor. | $\gamma_{M,\theta}$: 1.00

$\mathbf{k_y}$, $\mathbf{k_z}$, $\mathbf{k_{LT}}$: Interaction coefficients.

$$\mathbf{k_y} = 1 - \frac{\mu_y \cdot N_{Ed}}{\chi_y \cdot A \cdot f_{yd}} \le 3$$

$\mathbf{k_y}$: 1.00

$$\mathbf{k_z} = 1 - \frac{\mu_z \cdot N_{Ed}}{\chi_z \cdot A \cdot f_{yd}} \le 3$$

$\mathbf{k_z}$: 1.00

$$\mathbf{k_{LT}} = 1 - \frac{\mu_{LT} \cdot N_{Ed}}{\chi_z \cdot A \cdot f_{yd}} \le 1$$

$\mathbf{k_{LT}}$: 1.00

μ_y, μ_z, μ_{LT}: Auxiliary terms

$$\mu_y = \left(2 \cdot \beta_{M,y} - 5\right) \cdot \bar{\lambda}_y + 0.44 \cdot \beta_{M,y} + 0.29 \leq 0.8 \ ; \ \bar{\lambda}_y \leq 1.1$$

μ_y : *-0.28*

$$\mu_z = \left(1.2 \cdot \beta_{M,z} - 3\right) \cdot \bar{\lambda}_z + 0.71 \cdot \beta_{M,z} - 0.29 \leq 0.8$$

μ_z : *-0.03*

$$\mu_{LT} = 0{,}15 \cdot \bar{\lambda}_z \cdot \beta_{M,LT} - 0.15 \leq 0.9$$

μ_{LT} : *-0.11*

$\beta_{M,y}$, $\beta_{M,z}$, $\beta_{M,LT}$: Equivalent factors of uniform bending moment

$\beta_{M,y}$: *1.00*

$\beta_{M,z}$: *1.00*

$\beta_{M,LT}$: *1.00*

χ_{min}: Minimum reduction coefficient due to the connection betweenχ_y and χ_z. χ_{min} : *0.81*

χ_y, χ_z: Connection reduction coefficients, for the Y and Z axes, respectively.

χ_y : *0.81*

χ_z : *0.85*

χ_{LT}: Lateral reduction coefficient χ_{LT} : *1.00*

$\bar{\lambda}_y$, $\bar{\lambda}_z$: reduction of the space between the Y and Z axes, respectively.

$\bar{\lambda}_y$: *0.34*

$\bar{\lambda}_z$: *0.25*

Combined bending, axial and shear resistance – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.10, and EN 1993-1-2: 2005, Article 4)

It is not necessary to reduce the design bending and axial force resistance, as the buckling effect can be ignored due to shear. Additionally, the worst case design shear force $\mathbf{V_{Ed}}$ is less than or equal to 50% of the design shear resistance $\mathbf{V_{c,Rd}}$.

The worst case design forces occur for load combination SW

$$\mathbf{V_{Ed,y}} \leq \frac{V_{c,Rd,y}}{2}$$

0.00 kN ≤ 58.31 kN ✓

Where:

$V_{Ed,y}$: Rasti me i disfavorshem ne projektim per forcen prerese.	$V_{Ed,y}$:	*0.00*	kN
$V_{c,Rd,y}$: Design resistant shear force.	$V_{c,Rd,y}$:	*116.61*	kN

Torsional resistance – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.7, and EN 1993-1-2: 2005, Article 4)

The control does not continue, as there is no torsional moment

Combined Z shear and torsional resistance – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.7, and EN 1993-1-2: 2005, Article 4)

There is no interaction between the torsional moment and the shear force for any combination. Therefore the control is not done

Combined Y shear and torsional resistance – In case of fire (Eurocode 3 EN 1993-1-1: 2005, Article 6.2.7, and EN 1993-1-2: 2005, Article 4)

There is no interaction between the torsional moment and the shear force for any combination. Therefore the control is not done

16. RESULTS

The results of the calculations as well as the controls of the structural elements (slabs, beams, columns, walls, foundations) are given in the attached CD. On the basis of the results of dimensioning of the elements, their reinforcement was done as well as the detailing of each element in particular.

Also in the CD is given the modeling of the structure in Etabs.

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Tirane 2020